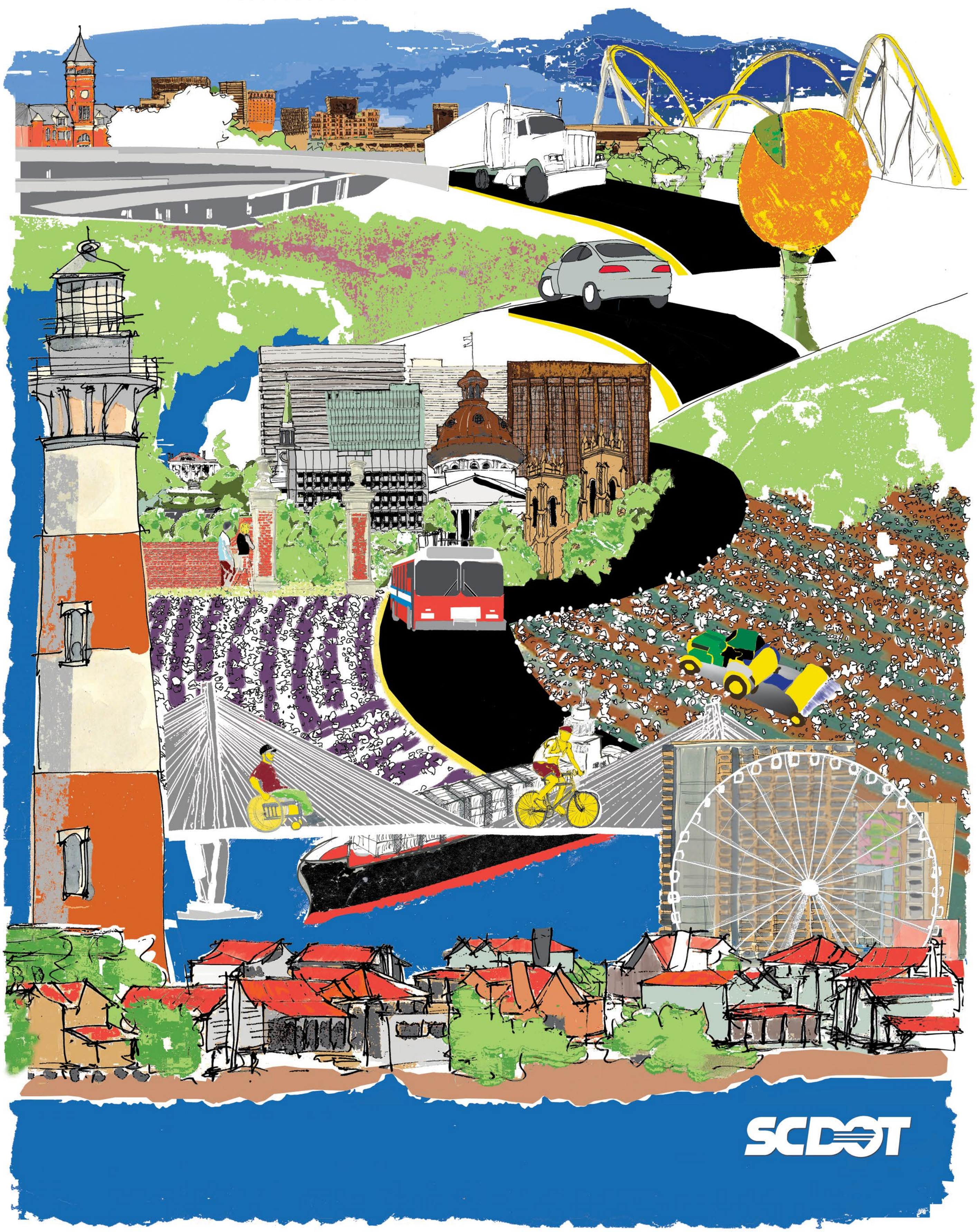
Roadway Design Manual

Connecting People And Places

March 2017



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PREFACE

The *South Carolina Roadway Design Manual* has been developed to provide uniform design practices for Department and consultant personnel preparing plans for projects on the state owned system. The designer should attempt to meet all criteria and practices presented in the *Manual*, while fulfilling the Department's operational, safety and context sensitive requirements. However, the *Manual* should not be considered a standard that must be met regardless of impacts. Designs will generally be made to values as high as are commensurate with roadway context. Values approaching the minimums herein should be used only where the use of higher values will result in unacceptable impacts on the economic, environmental, aesthetics, social and/or cultural resources of an area.

The *Manual* presents most of the information normally required in the design of a roadway project; however, it is impossible to address every situation that the designer will encounter. Therefore, designers must exercise good judgment on individual projects and, frequently, they must be innovative in their approach to roadway design. This may require, for example, additional research into highway literature.

The designer should consider constructability, maintenance, and operations when selecting the appropriate design. Inclusion of design features in this *Manual* does not constitute an agreement for the Department to maintain these features.

A SCDOT task force under the direction of the Preconstruction Support Section developed the *SCDOT Roadway Design Manual* with assistance from the engineering consultant firm of Roy Jorgensen Associates, Inc., and their subconsultant, Michael Baker International.

ROADWAY DESIGN MANUAL
Revision Process

The *South Carolina Roadway Design Manual* is intended to provide current design policies and procedures for use in developing State highway projects. To ensure that the *Manual* remains up-to-date and appropriately reflects changes in SCDOT's needs and applicable requirements, the *Manual* contents will be updated on an ongoing basis.

SCDOT will be responsible for evaluating changes in highway design literature (e.g., the issuance of new research publications, revisions to Federal regulations) and will ensure that those changes are appropriately addressed through the issuance of revisions to the *Manual*. It is important that users of the *Manual* inform SCDOT of any inconsistencies, errors, need for clarification or new ideas to support the goal of providing the best and most up-to-date information practical. A user who desires that a revision be considered for incorporation in the *Manual* should contact the Preconstruction Support Section.

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General Guidance

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Chapter 1
GENERAL GUIDANCE

1.1 ROADWAY DESIGN

The Department routinely issues policies and procedures that the designer must consider in the design of a roadway project. The following, present the memoranda, bulletins, etc., that are applicable to roadway design:

1. Engineering Directives. The SCDOT Deputy Secretary for Engineering sets engineering policy and direction, which requires compliance by the appropriate engineering divisions and all other providers of service to the Engineering Division (e.g., consultants, contractors). The Deputy Secretary for Engineering issues engineering directives containing the procedures for carrying out engineering policy.

2. Preconstruction Advisory Memoranda. The Director of Preconstruction issues Preconstruction Advisory Memoranda (PAM). PAMs provide policy and guidance for Preconstruction activities. Copies of the current PAMs can be obtained from the SCDOT internet website.

3. Preconstruction Design Memoranda. The Preconstruction Support Engineer issues Preconstruction Design Memoranda (PCDM). The purpose of Preconstruction Design Memoranda is to provide guidance on specific design issues. Copies of Preconstruction Design Memorandum can be obtained from the SCDOT internet site.

4. Design Memoranda. Design Memoranda are used to communicate updates/revisions to SCDOT design manuals and standards (drawings).

5. *SCDOT Roadway Design Manual*. The *SCDOT Roadway Design Manual* provides guidance on the design of highways for South Carolina. The *Manual's* geometric design treatments are based on AASHTO's *A Policy on Geometric Design of Highways and Streets* (Green Book), but tailored to the prevailing practices within South Carolina. In addition, the *Manual* is intended to clarify, where needed, specific presentations in the *Green Book* and to discuss geometric design information included in the *Green Book*.

6. AASHTO. AASHTO publishes many technical criteria and specifications pertinent to the design of highways and bridges.

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1.2 ACRONYMS

The following are commonly used acronyms for the major national agencies, publications and programs used in highway design:

1. AASHTO. American Association of State Highway and Transportation Officials
2. CFR. *Code of Federal Regulations*
3. CSS. Context Sensitive Solutions
4. FAST. *Fixing America's Surface Transportation Act*
5. FEMA. Federal Emergency Management Agency
6. FHWA. Federal Highway Administration
7. HCM. *Highway Capacity Manual*
8. HSIP. Highway Safety Improvement Program
9. HSM. *Highway Safety Manual*
10. ITE. Institute of Transportation Engineers
11. ISTEA. *Intermodal Surface Transportation Efficiency Act of 1991*
12. MAP-21. *Moving Ahead for Progress in the 21st Century Act*
13. MUTCD. *Manual* on *Uniform Traffic Control Devices*
14. NEPA. *National Environmental Policy Act*
15. NCHRP. National Cooperative Highway Research Program
16. NHS. National Highway System
17. SAFETEA-LU. *Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users*
18. SCDOT. South Carolina Department of Transportation
19. STP. Surface Transportation Program
20. TEA-21. *Transportation Equity Act for the 21st Century*
21. TRB. Transportation Research Board
22. TRR. Transportation Research Record
23. USC. *United States Code*
24. USDOT. United States Department of Transportation

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1.3 LAWS AND REGULATIONS GOVERNING DESIGN

1.3.1 Introduction

Highway designers are guided primarily by engineering principles and practices in making design calculations, developing design details and determining design requirements for their projects. In addition, designers must perform their work under the provisions or direction of Federal and State laws, regulations, rules, directives or other documents that have been enacted or issued. The following are some examples:

- *Code of Laws of South Carolina* and *SC Regulations*;
- *United States Code of Laws* (USC) and *Code of Federal Regulations* (CFR);
- South Carolina Department of Transportation, Engineering Directives;
- Federal Highway Administration policies and procedures;
- policies, criteria and guides of the American Association of State Highway and Transportation Officials (AASHTO); and
- criteria and guidance of other agencies (e.g., US Army Corps of Engineers, US Environmental Protection Agency, Coast Guard).

Copies of the United States and South Carolina Codes and Regulations can be found on the internet.

1.3.2 South Carolina Codes and Regulations

1.3.2.1 *Code of Laws of South Carolina*

Authority of the South Carolina Department of Transportation is derived from legislation. Over the years, various State statutes have been enacted that provide the background for administrative and technical controls and procedures that govern the Department. These statutes also establish sources and amounts of funds available to the Department for construction, maintenance and operation of the State Highway System. State laws sometimes are passed to provide necessary recognition and legal structure for the State to implement Federal laws. Various South Carolina statutes relating to the Interstate Highway System demonstrate this.

The major statutes of the *Code of Laws of South Carolina*, as amended, pertaining to the Department are contained in various volumes and titles of the law. Very general and selected areas of subjects, noted in parenthesis, that are pertinent to the Department are as follows:

- Title 12, “Taxation” (e.g., tax amounts, sources and distribution);
- Title 28, “Eminent Domain” (e.g., right-of-way condemnation requirements);
- Title 40, “Professions and Occupations” (e.g., engineers and land surveyors practices);

- Title 56, “Motor Vehicles” (e.g., regulating traffic; signs, signals and markings; vehicle sizes and weights);
- Title 57, “Highways, Bridges, and Ferries” (e.g., powers of Department, State Highway System, design references); and
- Title 58, “Public Utilities, Services, and Carriers” (e.g., utilities, railroads).

1.3.2.2 South Carolina Regulations

To implement the intent of the South Carolina laws applicable to highways and public transportation, the Department uses regulations under the provisions of the *South Carolina Administrative Procedures Act*. These regulations provide the administrative and operational direction for the Department to carry out its duties and responsibilities. The following provides a list of State agencies and their regulations, which may, in whole or in part, apply to SCDOT. These regulations are not necessarily all related to highway design.

- S.C. Department of Highways and Public Transportation, 63-10, et seq.;
- S.C. Department of Health and Environmental Control (DHEC), 61-1, et seq.;
- S.C. DHEC – Land Resources and Conservation Districts Division, 72-1, et seq.;
- S.C. DHEC – Coastal Division, 30-1, et seq.;
- S.C. Public Service Commission, 103-6, et seq.;
- S.C. Budget and Control Board, 19-101, et seq.;
- S.C. DNR – Water Resources Commissions, 121-8.0, et seq.;
- S.C. DNR – Wildlife and Marine Resources Divisions, 123-1, et seq.; and
- S.C. Department of Parks, Recreation and Tourism, 133-100, et seq.

1.3.3 <u>United States Code of Laws and Regulations</u>

The principal legislation governing highways is contained in Title 23 “Highways” of the *United States Code* (USC), the underlying authority for most of the regulations that govern the Federal Highway Administration. Title 49 “Transportation” of the USC and the regulations promulgated thereunder contain the laws that govern right-of-way acquisitions and procedures applicable to Federal projects.

As new highway Acts are passed, Title 23 of the USC is amended. The USC contains all Federal laws that have been codified and is amended annually to reflect changes to existing laws and the addition of new provisions. This codification embodies substantive provisions of law that Congress considers permanent that need not be reenacted in each new Highway Act. Each Highway Act specifies which sections of Title 23 are to be amended, repealed or added. Highway Acts are passed periodically. A Highway Act defines the rules for investment of Federal funds by establishing programs (e.g., National Highway Performance Program, Surface Transportation Program) that outline apportionment of those funds. These can be changed by subsequent Highway Acts.

The *Code of Federal Regulations* (CFR) contains regulations promulgated by the Federal government to implement Congressional Acts. These regulations provide the procedures necessary to implement the means, methods and procedures required by the law.

1.3.4 Federal Highway Administration Policy and Procedures

Federal regulations applicable to highways are implemented through the Federal Highway Administration (FHWA) and are set forth in 23 CFR.

Pursuant to Title 23, USC, Section 106(c), SCDOT may assume the responsibilities of the USDOT Secretary of Transportation under this title for design, plans, specifications, estimates, contract awards and inspections of projects unless SCDOT (for non-NHS routes) or the USDOT Secretary (for NHS routes) determines that such assumption is not appropriate.

SCDOT has agreed, through the Stewardship and Oversight Agreement with FHWA, that where it assumes delegated Title 23 oversight roles and approval responsibilities, it is responsible for ensuring that projects are developed and constructed in compliance with Federal requirements. FHWA may be consulted at any time regarding matters related to or affecting Federal requirements. SCDOT performs many activities during project development that require review and approval of FHWA. The Plan and a list of projects with specific oversight requirements are available on the SCDOT internet website.

1.3.5 American Association of State Highway and Transportation Officials

The American Association of State Highway and Transportation Officials (AASHTO) is an organization that facilitates the exchanges of information between the US Department of Transportation and the States. It functions as a forum for discussion of current transportation issues of concern and is frequently called upon by Congress to conduct surveys and provide data on transportation issues. AASHTO publishes many technical criteria and specifications pertinent to the design of highways and bridges. AASHTO revises and updates design publications periodically as new research and technology is developed.

1.3.6 Other Agencies

Directives and regulations issued by other Federal and State agencies influence the design of highways. Other agencies (e.g., US Army Corps of Engineers, US Environmental Protection Agency, US Coast Guard, South Carolina Department of Archives and History, South Carolina Department of Health and Environmental Control) also promulgate regulations that impact highway design.

The *National Environmental Policy Act of 1969* (NEPA) is the major Federal legislation governing environmental issues with regard to highway construction. The USDOT and FHWA promulgates regulations that establish the policies and procedures required for Federal-aid highway projects. Highway design is often influenced by environmental considerations (e.g., wetlands, archeological/historic sites, parks, recreational lands, endangered species habitats). Other Federal agencies have been empowered to issue regulations governing certain environmental concerns. Titles 23 and 40 of the *Code of Federal Regulations* contain the requirements, policies and procedures to be used in determining and processing environmental impacts of Federal-aid highway projects.

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1.4 QUALITY CONTROL/QUALITY ASSURANCE

Achieving quality in design is essential to providing a safe and efficient transportation system. In order to achieve quality, those responsible for the production of design plans should use consistent and ongoing quality control (QC) practices. Quality assurance (QA) verifies that consistent and ongoing quality control has occurred and that statewide consistency with designs and specifications has been achieved. The QA reviews will not take the place of QC practices.

However, quality is not the responsibility of a single group or unit, as it requires the total involvement of every employee involved in the project development process.

For additional information on SCDOT's quality control and quality assurance regulations, see PAM 4 "Preconstruction Quality Assurance Review Process" on the Department's internet site.

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1.5 REFERENCES

1. *Code of Laws of South Carolina* (1976), as amended (Volumes 5, 10, 14, 18, 19) and related *SC Regulations.*

2. *United States Code of Laws*; Title 23, Highways, as amended and related *Codes of Federal Regulations.*

3. *United States Code of Laws*; Title 49, Transportation, as amended and related *Codes of Federal Regulations.*

4. *United States Code of Laws*; Title 42, National Environmental Policy Act of 1969, as amended and related *Code of Federal Regulations.*

5. American Association of State Highway and Transportation Officials (AASHTO), various Manuals, Standards, Specifications, Guidelines (current editions).

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Chapter 2

CONTEXT SENSITIVE SOLUTIONS

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Chapter 2
CONTEXT SENSITIVE SOLUTIONS

The Context Sensitive Solutions (CSS) process is based on the concept that transportation projects should consider the context of their existence — not just the study area's physical boundaries. As defined on the Federal Highway Administration (FHWA) CSS website, CSS:

> *…is a collaborative, interdisciplinary approach that involves all stakeholders to develop a transportation facility that fits its physical setting and preserves scenic, aesthetic, historic and environmental resources, while maintaining safety and mobility. CSS is an approach that considers the total context within which a transportation improvement project will exist. CSS principles include the employment of early, continuous and meaningful involvement of the public and all stakeholders throughout the project development process.*

CSS recognizes how a highway or road is integrated within a community, can have far-reaching impacts (positive and negative) beyond its traffic or transportation function. This chapter provides guidance on the key principles and qualities of the CSS approach. It also includes references to additional information sources that provide further details and guidance on implementing the approach.

2.1 CSS RESOURCES

2.1.1 23 USC 109 "Standards"

Section 109(c)(1) of the *United States Code*, enacted by the 1995 *National Highway System Designation Act*, provides that a design for new construction, reconstruction, resurfacing (except for maintenance resurfacing), restoration or rehabilitation of highways on the National Highway System (other than highways also on the Interstate System) may take into account the constructed and natural environment of the area; the environmental, scenic, aesthetic, historic, community and preservation impacts of the activity; and access for other modes of transportation.

Section 109(c)(1), enacted by the *Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users* (SAFETEA-LU) and retained in the *Moving Ahead for Progress in the 21st Century Act* (MAP-21) authorizes the US Department of Transportation to consider the characteristics and qualities of CSS in establishing standards to be used on the National Highway System.

2.1.2 FHWA *Flexibility in Highway Design*

This 1997 FHWA publication provides guidance for highway engineers and project managers who want to learn more about the flexibility available when designing roads. The document was developed to provoke innovative thinking considering the scenic, historic, aesthetic and other cultural values, along with the safety and mobility needs of the highway transportation system.

2.1.3 NCHRP Report 480 *A Guide to Best Practices for Achieving Context Sensitive Solutions*

This 2003 publication provides detailed guidance for implementing the CSS approach. It includes sections on:

- effective decision making,
- reflecting community values,
- achieving environmental sensitivity,
- ensuring safe and feasible solutions,
- organizational needs, and
- case studies in Context Sensitive Design/Context Sensitive Solutions.

2.1.4 NCHRP Report 642 *Quantifying the Benefits of Context Sensitive Solutions*

This 2009 publication provides guidelines for determining the benefits of CSS. The key topics discussed include:

- introduction to the approach of benefit quantifications for projects;
- application requirements, standardize methods and data collection tools;
- project evaluation example illustrating a complete application;
- the action principles of CSS project development; and
- the principle-associated benefits of CSS.

2.1.5 AASHTO *Guide for Achieving Flexibility in Highway Design*

This 2004 AASHTO publication provides guidance for designers on how to think flexibly, recognize the many choices and options, and arrive at the best solution for a particular context.

2.1.6 FHWA CSS Website

This website includes links to information explaining CSS and its history, current CSS-related activities, guidance on CSS issues, and numerous other CSS resources.

2.1.7 Context Sensitive Solutions Website

The Context Sensitive Solutions website provides a Context Sensitive Solutions Resource Center that includes information and links for a broad range of CSS topics. It was created by Project for Public Spaces in collaboration with Scenic America to assist FHWA in integrating CSS into project planning, development and implementation.

2.1.8 Center for Environmental Excellence by AASHTO Website

This website addresses the following topics:

- Background (What is CSS? Where Did CSS Come From?)

- Why is CSS Important to Transportation Agencies?
- What Steps Can Help Institutionalize and Integrate CSS?
- Where Does CSS Apply in Program and Project Delivery?
- Links to CSS-Related Laws, Policies and Guidance.

2.1.9 SCDOT Traffic Calming Guidelines

This publication provides guidance to SCDOT personnel and local governments concerning traffic calming. The document details eligibility requirements, application forms, various traffic calming measures, construction specifications and web links to traffic calming resources.

This document is located on the Department's internet site.

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2.2 DESIGN TREATMENTS

This section highlights appropriate CSS treatments for both construction and maintenance elements of a road corridor and is intended to guide designers and planners working on a project. These suggested treatments do not include all potential solutions for the multitude of projects designed and constructed. They are presented as guidance that can be used for improvements or providing enhancement. The designer needs to use engineering judgement in determining the appropriate action for the design.

The suggested treatments provide guidance by exploring design solutions that are context-sensitive and consider stakeholder interest. Appropriate treatments for each project should result from a process where cost, regulatory considerations, design guidelines and safety are taken into account. Roadside elements can dramatically affect a roadway's character. When working within roadways, mobility and safety are the primary transportation concerns, while context, visual quality and traveler experience are equally important to stakeholders. Designers should become acquainted with best practices in transportation to help identify the right solutions and consider flexibility when developing an appropriate solution for the task. Review the resources listed in Section 2.1 for additional options.

2.2.1 Vision

A project vision, including purpose and need, should be developed and clearly documented with the involvement of project stakeholders early in the process. This vision should then guide the project development decisions. Project team members, from project planning, design, rights of way, construction, maintenance and traffic should appreciate the importance of each function and agree to the project vision in order to successfully achieve that vision.

The project purpose can include other secondary, but important factors to the community that may influence the design, choice or success of the solutions. For example, the aesthetic appearance of safety improvements in the transportation system can affect the use, and hence, the intended benefit of such features as crosswalks or bridges.

2.2.2 Public Involvement

The cornerstone of successful CSS is public involvement. Effective public involvement encourages the exploration of issues from a variety of perspectives. Stakeholders need to be identified at the beginning and during the planning, programming and development processes, and be involved throughout these processes. Open collaboration and exchange of information and concerns between the transportation planners, designers and stakeholders promotes buy-in to project outcomes and trust among stakeholders.

This process includes talking, listening, teaching and learning. While projects are not expected to be unanimously endorsed by every citizen, the Department is committed to providing users with projects that meet their needs and fit into their communities. Good communication throughout the project and using appropriate tools to develop consensus among project stakeholders helps achieve support.

2.2.3 Design Exceptions

Designers are faced with many complex tradeoffs when designing highways and streets. A good design balances cost, safety, mobility, social and environmental impacts, and the needs of a wide variety of roadway users. Good design is also context-sensitive — resulting in streets and highways that are in harmony with the natural and social environments through which they pass.

It must be recognized, however, that to achieve the balance described above, it is not always possible to meet design criteria. Establishing design criteria that cover every possible situation, each with a unique set of constraints and objectives, is not possible. On occasion, designers encounter situations in which the appropriate solution may suggest that using a design value or dimension outside the normal range of practice is necessary. Arriving at this conclusion requires the designer to understand how design criteria affect safety and operations. For many situations, there is sufficient flexibility within the design criteria to achieve a balanced design and still meet minimum values. However, when this is not possible, a design exception may be the best option. See Section 3.2 for guidance on design exceptions.

2.2.4 Flexibility

The setting and character of the area, the values of the community, the needs of the highway users, and the challenges and opportunities are unique factors that designers must consider with each highway project. Whether the design to be developed is for a modest safety improvement or 10 miles of new-location highway, there are no patented solutions. For each potential project, designers are faced with the task of balancing the need for the highway improvement with the need to safely integrate the design into the surrounding natural and human environments. Design elements to consider include road alignment, roadside structures, sidewalks, shared-use paths, landscaped medians and street trees, traffic signs, utility facilities, site furniture and bridge design.

This *Manual* recognizes the need for flexibility and provides that flexibility (e.g., low-volume rural roads or residential areas versus higher volume rural or urban facilities). The formulation of these values demonstrates considerable flexibility.

2.2.5 Central Business District

A central business district (CBD) is the commercial, business and transportation network center of a city or town. In larger cities, it is often synonymous with the city's "financial district." Geographically, it often coincides with the "city center" or "downtown," but the two concepts are separate. Many cities have a central business district that is located away from its commercial or cultural city center or downtown. CBDs typically have development immediately adjacent to the right-of-way line with sidewalks from the curb to the business fronts.

2.2.6 Driver Expectancy

Driver expectancy relates to the readiness of the driver to respond to events or the presentation of information. It can be defined as an inclination, based on previous experience, to respond in a set manner to a roadway or traffic situation. It should be stressed that the initial response is to the expected situation rather than the actual one.

There are certain elements in the design of various components of the roadway that particularly affect design consistency, driver expectancy and vehicular operation. These components include horizontal and vertical alignment, embankments and cut slopes, shoulders, crown and cross slopes, superelevation, bridge widths, signing, delineation and guardrail.

2.2.7 Design Consistency

Design consistency is achieved when the geometric features of the roadway are consistent with the operational characteristics expected by the driver. Inconsistencies normally relate to:

- changes in design speed,
- changes in cross section, and/or
- incompatibility in geometry and operational requirements.

Variations in design speed may occur on a given stretch of roadway where portions of the highway have been constructed as separate projects over an extended time period. Inconsistencies may include changes in criteria or SCDOT policy, reclassification of the facility or financial feasibility.

Driver expectancies are formed through experience and training. The successful response to situations that generally occur in the same way is an important part of the driver's store of knowledge. The following are two major types of design inconsistencies relative to cross sections:

1. Service Inconsistencies. Service inconsistencies may include:

    - cross-sectional differences within a given section of highway that are untypical of the area (e.g., one bridge, among many, that does not have full shoulder widths);

    - a short two-lane section on a multilane segment of a highway;

    - a single lane drop on a section of highway that does not have any other lane drops; or

    - a left exit on a freeway where all other exits are from the right.

2. Alignment Inconsistency. Cross-sectional inconsistencies are usually the result of upgrading a highway cross section without upgrading the alignment. Pavements may be widened and shoulders added on an existing two-lane highway. The wider cross section might convey a conflicting message to the driver and lead to an inappropriate expectancy based on the visual aspects of the cross section. However, widening alone can measurably improve the safety characteristics of a road, particularly on very narrow, low-volume roads. Designers should be aware of potential inconsistencies that frequently can be overcome with relatively low-cost treatments. Pavement markings, warning signs and delineation devices can be very helpful to the driver when roads are widened on existing alignments.

Incompatibility in geometric and operational requirements may result even when geometric components are appropriately selected. For example, a direct entry ramp is designed to permit vehicular entry into the stream of traffic without coming to a complete stop; however, the vehicle is forced to stop when a gap in the through traffic stream is not immediately available.

2.2.8 Lane Width

Lane width has an influence on the safety and comfort of the driver. The physical dimensions of cars and trucks, speeds, highway type and vehicle type influence the width of the travel lanes. The normal through lane width is 10 feet to 14 feet, and for auxiliary lanes 9 feet to 12 feet, depending on percent of truck usage. Lane width ranges are provided in Chapters 14, 15, 16, 17 and 18.

Wider lane widths are typically associated with higher speed roadways (e.g., freeways, arterials). As speed and traffic volumes increase, additional lane width is desirable to accommodate the variations in lateral placement of the vehicle within the lane. Greater lane widths better accommodate wider vehicles in the traffic stream (e.g., trucks, buses, recreational vehicles). Wider lane widths also marginally increase the capacity of the roadway.

For lower speeds, lower volume roads and streets with little or no truck traffic, through lane widths as narrow as 10 feet may be acceptable; lane widths less than 12 feet are considered adequate for a wide range of volume, speed and other conditions. Design for lane width should include consideration of the horizontal alignment. Adequate lane width is very important along horizontal curves because vehicles off-track, which means that their paths exceed the width of the vehicle. See Section 5.2.6. They require additional room to avoid encroaching into the opposing traffic, adjacent travel lanes and/or the shoulder, which may also be used by pedestrians and/or bicyclists. Increased lane width reduces the demands placed on the motorists by reducing the amount of concentration needed to stay within the travel lane.

In urban areas and along rural routes that pass through urban settings, narrower lane widths may be appropriate. For these locations, space is limited and lower speeds are desired. Narrower lane widths for urban streets lessen pedestrian crossing distances, enable the provision for on-street parking and transit stops, and enable the development of left-turn lanes for safety. Lesser widths also tend to encourage lower speeds, an outcome that may be desirable in urban areas. In considering the use of narrower lanes, designers should recognize that narrow travel lanes reduce vehicle separation from other vehicles and bicyclists.

2.2.9 Shoulder Width

Shoulders, whether paved or unpaved, serve a variety of functions. Shoulders:

- provide structural support for the traveled way;
- provide space for emergency storage of disabled vehicles, enforcement and maintenance activities;
- provide an area for drivers to maneuver to avoid crashes;
- improve bicycle accommodation;
- increase safety by providing a stable, clear recovery area for drivers who have left the travel lane;
- improve stopping sight distance;
- store and carry water; and
- improve capacity by increasing driver comfort.

Shoulder widths typically vary from 2 feet to 12 feet. Regardless of width and surfacing, shoulders should be flush with the roadway surface and sufficiently stable to support vehicular use in all kinds of weather without rutting. Criteria for shoulder widths are provided in Chapters 14, 15, 16, 17 and 18.

Where a full-width shoulder cannot be achieved, the designer should strive to provide as wide a shoulder as practical that meets its functional requirements. Major functions of the shoulder are to provide sight distance and serve as part of the clear zone. Mitigating a narrow shoulder can include the provision of a wider clear zone or flatter side slope to partially counteract the loss of the shoulder. The use of traversable ditch designs may also be appropriate where narrow shoulders are used. Sight distance can be mitigated by revising cut slopes, shifting horizontal alignment, revising vertical alignment or geometric improvements.

Another important function is the storage of disabled or stopped vehicles. If a full, continuous shoulder is not possible, designers should at least seek to provide intermittent full-width turnouts, especially on higher-volume, high-speed roads. The provision for full or at least operationally functional shoulder widths associated with vehicle refuge and law enforcement supports incident management.

2.2.10 Road Alignment and Design Speed

The horizontal and vertical alignment of a roadway greatly affects the driver's experience and contributes to the scenic features of the corridor. Many South Carolina roadways are characterized by the road's curving nature as it follows the topography of the natural landscape. In some circumstances, where vehicles move faster than the roadway's ability to safely accommodate them, these undulating roadways may have a higher number of crashes.

In addition to safety concerns, roads historically evolve in response to an increase of use. Alignments are straightened for improved visibility, shoulders are paved and roadways are widened to accommodate turning or passing lanes. While these changes are made to improve mobility and safety, they can affect the original visual appeal of the roadway and detract from the driver's experience.

While evaluating the safety and mobility considerations associated with road realignment is vital, maintaining character defined features is equally important.

Because many current South Carolina roadways are in rural areas where traffic volumes do not approach the capacity of the roadway, safety is the driving force behind most alignment decisions along those roadways. When safety is an issue, there are two basic ways for designers to consider the relationship between operating speed and road design:

1. Traditional Engineering Approach. The road's existing horizontal alignment, vertical alignment and/or typical section are inadequate to safely convey traffic at anticipated volumes and speeds. For this reason, the road should be straightened and/ or widened to enhance safety, which may affect the valued character-defining features of the roadway.

2. Alternative Approach. Traffic is traveling too fast to negotiate safely the roadway's alignment and width. For this reason, the designer should consider reducing operating speeds to enhance safety as well as preserve roadway character-defining features.

The key to both approaches is the selection of an appropriate design speed. Design speed is arguably the most important design control used in selecting standards for the design of a roadway segment. The appropriate target speed should be based on land use conditions, building densities, environmental context and the needs of users. See Section 3.5.2 for guidance on selecting design speeds. Designers should seek consistency among all aspects of the roadway, its context and the chosen design speed.

The core principle is that the design speed should not be lower than the anticipated operating speed. However, selection of the anticipated operating speed is critical. It need not (and, in fact, should not) be based solely on the current speed limit or existing measured speed. Future operating speed, for example, can be safely influenced by the design of the roadway.

Selection of a lower design speed within the parameters of terrain, land use and functional classification may reduce the need for adjustments to horizontal alignment, vertical alignment and typical section. This can reduce impacts and project costs while still ensuring appropriate roadway safety and capacity while preserving the character-defining features of the roadway.

Along local streets, the designer may also include traffic calming measures. Traffic calming measures tend to be more appropriate along urban or small-town portions of roadways, and their aesthetic effects on the surrounding landscape should be considered. In contrast with passive techniques (e.g., lane widths) traffic calming measures actively reduce the speed of vehicles through horizontal and vertical geometry. For additional guidance, see the *SCDOT Traffic Calming Guidelines*.

2.2.11 Roadside Barriers

Wide varieties of traffic barriers are available for installation along highways and streets, including both longitudinal barriers and crash cushions. Design of traffic barriers is an important detail that contributes to the overall look of the roadway; therefore, in addition to safety, the selection of an appropriate barrier design should include aesthetic considerations. Because aesthetic considerations are usually a factor, many barriers are designed to add to the visual quality while meeting crash test criteria for facilities with truck traffic. Given these options, designers must balance decisions based on safety, cost and aesthetics. For additional guidance on roadside barriers, see the AASHTO *Roadside Design Guide* and *SCDOT Standard Drawings.*

2.2.12 Bridges, Walls and Other Structures

Bridges and small structures can contribute to or detract from a roadway's character and quality. If an existing bridge or small structure is considered a character-defining feature of the roadway, it should be preserved through maintenance, rehabilitation and repair, if possible. When a bridge must be replaced, compatibility can be achieved by replacing the structure in-kind or by reconstructing a bridge with similar detail. If, however, the bridge detracts from the roadway's character, a replacement bridge can enhance the road if a design more compatible with the character of the roadway and its users is selected.

All structural design should take into account the context of the landscape and reflect its historic, rural or urban character. As viewed in its context, form is most affected by the geometry and the type of bridge structure chosen. The designer should choose materials and colors that are complementary to the landscape while maintaining compliance with applicable design criteria.

Color and texture can be used to reduce or enhance the visual contrast depending upon design goals and can be applied to multiple stages of design.

2.2.13 Bicycle Facilities

Bicycles are a viable mode of transportation in South Carolina, both for commuting and recreation. SCDOT practice is to consider bicycles and pedestrians on all South Carolina roadways. This ensures that system modifications are routinely planned, designed, constructed, operated and maintained in a way that enables safe and efficient access for all users. The result should be a system for all users that is comprehensive, integrated, connected, safe and efficient allowing users to choose among different transportation modes, both motorized and non-motorized.

Accommodating bicycles on roadways often presents challenges that can result in widening of roadways, potentially altering character-defining features. For example, an important feature of many roadways is the narrow two-lane cross section through rural areas. Projects along roadways that have scenic intrinsic qualities should strive to preserve this narrow pavement and more intimate and pastoral scale while still accommodating bicyclists.

Designers should be familiar with current standards and guidance for bicycles and incorporate them into their projects. This can be particularly important in roadway projects, where creative design is necessary to allocate limited roadway and/or right of way for all modes of travel. Section 11.11 and AASHTO *Guide to the Development of Bicycle Facilities* provide guidelines for bicycle facilities. In addition, the ITE document *Designing Walkable Urban Thoroughfares: A Context Sensitive Approach* provides guidance on the tradeoffs among all modes of travel. This document is especially valuable in understanding what options exist in terms of widths for motor vehicle lanes, bike lanes, shared lanes, shared-use paths and sidewalks.

2.2.14 Pedestrian Environment

In keeping with South Carolina's practice, the designer must consider the needs of pedestrians along all State roadways. Sidewalks, where provided, are not just pedestrian thoroughfares; they are social places in communities serving adjacent land uses. The surrounding context, particularly important in urban roadway projects is the sidewalk's physical condition (e.g., existing grade, access points, cross slope, width, materials) and location (e.g., historic town, urban downtown, residential area, nature preserve).

Sidewalks should accommodate pedestrians of all ages and abilities, with attention given to locating pedestrian amenities that logically direct people to desired destinations in a safe and attractive environment. Sidewalk design and maintenance should respond to the context and address the full variety of functions the sidewalk will serve. Generally, sidewalks in urban areas should provide opportunities for planting buffers, bus stops, signs and street trees. These users will require access to adjacent shops and services as well as on-street parking and public transit. In rural/suburban areas, sidewalks usually serve children traveling to school and recreational activities (e.g., walking, jogging, biking).

In suburban and rural areas where land uses are not located near the back of the sidewalk, it is more common to find a pedestrian zone separated from the roadway by the shoulder or a grass strip. Designers should consider the types of pedestrians who use the sidewalk (e.g., ranges of age, mobility, ability) and how long pedestrians use the sidewalk — long-term (gathering spaces), short-term (mass transit stops) or transition (walking through).

Section 7.3.3 provides further guidance on sidewalks.

2.2.15 Landscape

Trees and other vegetation play a vital role in defining the spatial relationship of a corridor. They often represent an indigenous or designed landscape and enhance the aesthetic quality of the roadway. A rolling open field or a canopy-covered street can contribute to a memorable travel experience while the aesthetics of changing seasons often attract visitors year round. However, the treatment of trees along a roadway may pose safety and aesthetic conflict between designers and stakeholders, especially for trees identified as vital character-defining features requiring protection and preservation. *Note: mature trees greater than four inches in diameter inside the clear zone are considered fixed objects that may require removal.*

CSS encourage designers to explore flexible alternatives that augment the roadway's intrinsic qualities, reflect community values and meet engineering requirements for safety and mobility. Along roadways where trees are identified as important features, the designer should consider traffic characteristics and safety concerns by using design minimums, lower design speeds and/or minimum clear zone widths. Application of alternative techniques (e.g., modified road alignment, adding curb, protective barriers, roadway lighting, pavement striping, warning signs, shoulder rumble strips) is also encouraged. The desire for plantings is common on urban, context sensitive projects. See the Department's website for landscaping guidance.

2.3 REFERENCES

1. "Context Sensitive Solutions (CSS)," FHWA Website, 2012.

2. NCHRP Report 480, *A Guide to Best Practices for Achieving Context Sensitive Solutions*, Transportation Research Board, 2002.

3. NCHRP Synthesis 422, *Trade-Off Considerations in Highway Geometric Design*, Transportation Research Board, 2011.

4. NCHRP Report 642, *Quantifying the Benefits of Context Sensitive Solutions*, Transportation Research Board, 2009.

5. Title 23, Section 109 "Standards," *United States Code*, 2012.

6. *Flexibility in Highway Design*, FHWA-PD-97-062, Federal Highway Administration, 1997.

7. *A Guide for Achieving Flexibility in Highway Design*, AASHTO, May 2004.

8. "Context Sensitive Solutions.org," Project for Public Spaces Website, 2012.

9. "Center for Environmental Excellence," AASHTO Website, 2012.

10. *Guide for the Planning, Design, and Operations of Pedestrian Facilities*, AASHTO, July 2004.

11. *Designing Walkable Urban Thoroughfares: A Context Sensitive Approach, An ITE Recommended Practice*, Institution of Transportation Engineers, March 2010.

12. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

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Chapter 3

BASIC DESIGN CONTROLS

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 3
BASIC DESIGN CONTROLS

Road design is predicated on many basic controls that reflect the overall objective of the highway facility and identify the basic purpose of the highway project. This chapter presents these basic controls that impact highway design. The application of these items to a project will impact all elements of highway design.

3.1 QUALIFYING WORDS

Many qualifying words are used in highway design and in this *Manual*. For consistency and uniformity in the application of various design criteria, the following definitions apply:

1. <u>Shall, Require, Will, Must</u>. A mandatory condition. Designers are obligated to adhere to the criteria and applications presented in this context or to perform the evaluation indicated. For the application of geometric design criteria, this *Manual* limits the use of these words.

2. <u>Should, Recommend</u>. An advisory condition. Designers are strongly encouraged to follow the criteria and guidance presented in this context, unless there is reasonable justification not to do so.

3. <u>May, Could, Can, Suggest</u>. A permissive condition. Designers are allowed to apply individual judgment and discretion to the criteria when presented in this context. The decision will be based on a case-by-case assessment.

4. <u>Desirable, Preferred</u>. An indication that the designer should meet minimum criteria unless a lesser value is reasonably justified.

5. <u>Ideal</u>. Indicating a standard of perfection (e.g., traffic capacity under ideal conditions).

6. <u>Minimum, Maximum, Upper, Lower (Limits)</u>. Representative of generally accepted limits within the design community, but not necessarily suggesting that these limits are inviolable. However, where the criteria presented in this context will not be met, the designer will, in many cases, need approval.

7. <u>Practical, Feasible, Cost-Effective, Reasonable</u>. Advising the designer that the decision to apply the design criteria should be based on a subjective analysis of the anticipated benefits and costs associated with the impacts of the decision. No formal analysis (e.g., cost-effectiveness analysis) is intended, unless otherwise stated.

8. <u>Possible</u>. Indicating that which can be accomplished. Because of its rather restrictive implication, the term possible is rarely used in this *Manual* for the application of design criteria.

9. <u>Significant, Major</u>. Indicating that the consequences from a given action are obvious to most observers and, in many cases, can be readily measured.

10. Insignificant, Minor. Indicating that the consequences from a given action are relatively small and not an important factor in the decision making for highway design.

11. Warranted, Justified. Indicating that some well-accepted threshold or set of conditions has been met. As used in this *Manual*, warranted or justified may apply to either objective or subjective evaluations. Note that, once the warranting threshold has been met, this is an indication that the designer should consider and evaluate the design treatment — not that the design treatment is automatically required.

12. Standard. Indicating a design value that should not be violated without fully understanding the consequences. This suggestion is generally inconsistent with geometric design criteria. Therefore, the term standard is rarely used in this *Manual* to apply to geometric design criteria.

13. Standard Practice. Indicating an unwritten, preferred practice established by procedure.

14. Guideline. Indicating a design value that establishes an approximate threshold that should be met if considered practical.

15. Criteria. A term typically used to apply to design values, usually with no suggestion on the criticality of the design value. Because of its basically neutral implication, this *Manual* frequently uses criteria to refer to the design values presented.

16. Typical. Indicating a design practice that is most often used in application.

17. Acceptable. Design criteria that may not meet desirable values, but is considered reasonable for design purposes.

18. Policy. A SCDOT requirement the designer must follow. A high-level overall plan embracing general goals and acceptable procedures.

3.2 ADHERENCE TO GEOMETRIC DESIGN CRITERIA

The *South Carolina Roadway Design Manual* presents numerous criteria on road design for application on individual road design projects. The general intent of the South Carolina Department of Transportation is that all road design criteria in this *Manual* should be met and, wherever practical, the proposed design should exceed the minimum criteria. The Department's intent is to provide a highway system that meets the transportation needs of the State while ensuring an acceptable level of safety, comfort and convenience for the traveling public.

Recognizing that meeting the minimum criteria may not always be practical, the Department has established guidelines for design exceptions and design variances. See the Department's guidelines for further information.

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3.3 PROJECT SCOPE OF WORK

The project scope of work will reflect the basic intent of the highway project and will determine the overall level of highway improvement. This decision, in combination with the highway functional classification (see Section 3.4.1), will determine which criteria in the *Manual* apply to the geometric design of the project. The following provides general definitions for the project scope of work, and references to the applicable chapters for the design criteria based on the project scope of work.

3.3.1 New Construction

Generally, new construction is defined as horizontal and vertical alignment on new location. The development is based on a 20-year design period from the date the PS&E package is completed. The project will be logical in scope and have logical termini. Where a facility is on new location and has a new alignment, it is considered new construction. In addition, new construction also includes any new intersection or interchange that falls within the project limits of a new or existing highway mainline or is relocated to a new point of intersection. Chapter 14 "Local Roads and Streets," Chapter 15 "Collector Roads and Streets," Chapter 16 "Rural and Urban Arterials" and Chapter 17 "Freeways" present SCDOT's criteria for new construction.

3.3.2 Reconstruction

Reconstruction of an existing highway will typically include the addition of travel lanes, reconstructing of the existing horizontal and vertical alignment, widening the roadway and flattening side slopes, but the highway will remain essentially within the existing highway corridor. These projects will usually require some right-of-way acquisitions. The primary reasons for reconstructing an existing highway are because the facility cannot accommodate its current or future traffic demands, the existing alignment or cross section is deficient and/or the service life of the pavement has been exceeded. In addition, any intersection that falls within the limits of a reconstruction project will be reconstructed as needed.

Because of the significant level of work for reconstruction, the design of the project generally will be determined by the criteria for new construction based on a 20-year design period. The criteria in Chapters 14 through 17 will apply to reconstruction projects.

3.3.3 3R Projects (Non-Freeways)

A significant percentage of the Department's current and future highway program involves work on existing highways. The Department's responsibility is to realize the greatest overall benefit from the available funds. Therefore, the geometric design of projects on existing highways must be viewed from a different perspective than the design of new construction/reconstruction projects. Resurfacing, restoration and rehabilitation (3R) projects are often initiated for reasons other than geometric design deficiencies (e.g., pavement deterioration), and they must often be designed within restrictive right of way, and financial and environmental constraints. Therefore, the design criteria for new construction are often not attainable without major and, frequently, unacceptable adverse impacts. At the same time, however, the Department must take the opportunity to make cost-effective, practical improvements to the geometric design of existing highways and streets. See Chapter 18 “3R Projects (Non-Freeways)” for further guidance on the goals and objectives for 3R projects.

3R work on the mainline or at an intersection is typically work within the existing alignment. However, right-of-way acquisition is sometimes justified for flattening slopes, changes in horizontal alignment, changes in vertical profile and safety enhancements.

The overall objective of a 3R non-freeway project is to perform work necessary to return the highway to a condition of acceptable structural and/or functional adequacy. 3R projects may include any number of the following types of improvements:

- providing pavement resurfacing, pavement rehabilitation and/or pavement reconstruction;
- providing lane and/or shoulder widening (without adding through lanes);
- paving shoulders;
- correcting skid hazards;
- adding a two-way, left-turn lane (TWLTL);
- adding a bike lane;
- providing intersection improvements (e.g., adding or extending turn lanes, flattening turning radii, adding channelization, realigning minor road, improving corner sight distance);
- flattening a horizontal or vertical curve;
- adding curb and gutter to an existing urban street;
- removing, widening and/or resurfacing parking lanes;
- upgrading at-grade highway/railroad crossings;
- revising the location, spacing or design of existing driveways along the mainline;
- roadway approach work associated with a bridge rehabilitation and/or widening;
- upgrading bridge rails;
- upgrading guardrail and other roadside safety appurtenances to meet current criteria;
- relocating utility poles;
- removing, providing and/or upgrading traffic control devices;
- adjusting the roadside clear zone;
- flattening side slopes;

- providing drainage improvements;
- adding or removing transit stops;
- implementing improvements to meet the Department's accessibility criteria (e.g., sidewalks and sidewalk curb ramps);
- upgrading to current access management policies; and/or
- incorporating multimodal operations.

3.3.4 Preventive Maintenance Projects

For new construction/reconstruction projects, use Chapters 14 through 17 and the applicable criteria provided elsewhere in this *Manual*. See the Department's Memorandum of Agreement for Federal-Aid Preventive Maintenance Projects for guidance on approved preventive maintenance activities.

3.3.5 Spot Improvements

Spot improvements are intended to correct an identified deficiency at an isolated location. The deficiency may be related to structural, geometric, safety, drainage or traffic control problems. These projects are not intended to provide a general upgrading of the highway, as are projects categorized as new construction, reconstruction or 3R. For these reasons, a flexible approach is necessary to determine the appropriate geometric design criteria that will apply to the spot improvement.

3.3.5.1 Highway Safety Improvement Projects

These projects are intended to provide cost-effective improvements to sites identified in the Department's Strategic Highway Safety Plan. Safety projects are improvements intended to correct isolated highway deficiencies at locations where high crash or severity rates can be substantiated or where a high potential for crashes exists. The process of identifying hazardous locations, plus the procedures for selecting and developing a prioritized listing of projects for improvements, is discussed in the SCDOT *Highway Safety Improvement Program Manual*.

Roadway and bridge deficiencies may be related to structural, geometric, safety, drainage or traffic control problems. It is not the intent of the safety improvement project to provide for general upgrading of highways. The selected criteria should be based on sound engineering assessments of conditions at the particular site. In general, the objective is to improve the problem area to a level of driver expectancy equivalent to that of adjacent sections of roadway.

Safety improvements may include intersection improvements, flattening horizontal and vertical curves, replacing or rehabilitating obsolete bridge rails and guardrails, flattening side slopes, removing roadside obstacles, resurfacing, improving railroad crossings, correcting pavement drop-offs, improving signing, pavement markings and/or other traffic control devices.

Safety projects are generally developed and managed by the Traffic Engineering Division.

3.3.5.2 Special Projects

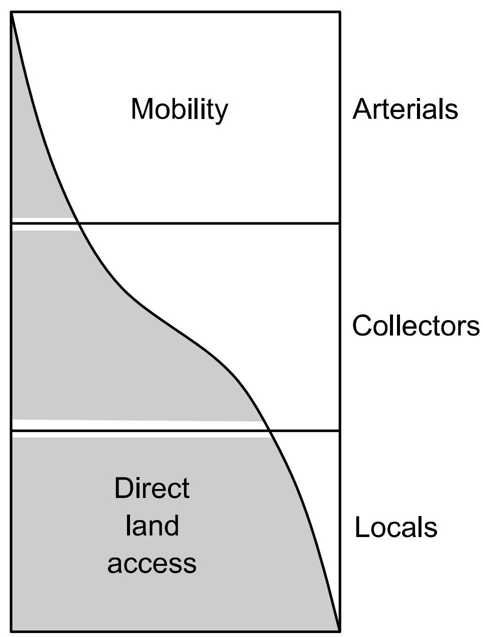
These are projects that generally do not qualify as a safety improvement project as defined in Section 3.3.5.1 and usually require State funding. Special projects are often site specific and design criteria must be tailored to meet the needs of the project. Typical projects may include adding turn lanes, providing by-pass lanes, removing roadside obstacles, etc. In some instances, special projects are implemented with the Department's Maintenance forces. These improvements should be coordinated with the Director of Maintenance and the District Engineering Administrator.

3.4 HIGHWAY SYSTEMS

3.4.1 Functional Classification

3.4.1.1 Relationship to Design

The functional classification concept is one of the most important determining factors in highway design. In this concept, highways are grouped by the character of service they provide. Functional classification recognizes that the public highway network in South Carolina serves two basic and often conflicting functions — travel mobility and access to property. See Figure 3.4-A. Each highway or street will provide varying levels of access and mobility, depending upon its intended service. In the functional classification scheme, the overall objective is that the highway system, when viewed in its entirety, will yield an optimum balance between its access and mobility purposes. If this objective is achieved, the benefits to the traveling public will be maximized.



MOBILITY VERSUS ACCESS
Figure 3.4-A

The functional classification system provides the guidelines for determining the geometric design of individual highways and streets. Once the function of the highway facility is defined, the designer can select an appropriate design speed, roadway width, roadside safety elements, amenities and other design values. The *SCDOT Roadway Design Manual* is based upon this systematic concept to determine geometric design.

Road Data Services has functionally classified all public roads and streets within South Carolina that are maintained by the Department. For highway design, it is necessary to identify the predicted functional class of the road or street for the selected design year (e.g., 20 years beyond the project completion date). Road Data Services will provide the current functional classification. The designer is responsible for predicting the future functional classification.

There are three general categories within the functional classification system — arterials, collectors, and local roads and streets. The following sections provide brief definitions for these categories. The AASHTO *A Policy on Geometric Design of Highways and Streets* provides detailed information on functional classification.

3.4.1.2 Arterials

Arterial highways are characterized by a capacity to quickly move relatively large volumes of traffic, but are often impacted by their service to abutting properties. The arterial functional class is subdivided into principal and minor categories for rural and urban areas:

1. Principal Arterials. In both rural and urban areas, the principal arterials provide the highest traffic volumes and the greatest trip lengths. The designer should review the project scope of work and planning documents to determine which of the following principal arterials should be used in the design and identify its corresponding criteria:

    a. Freeways. The freeway is the highest level of principal arterial. These facilities are characterized by full control of access, high design speeds and a high level of driver comfort and safety. For these reasons, freeways are considered a special type of highway within the functional classification system, and separate design criteria have been developed for them.

    b. Urban/Rural Arterials. These facilities are usually two or four lanes with or without a median. Partial control of access is desirable along these facilities. A high level of geometric design is desirable to move the high traffic volumes quickly and efficiently through an area.

2. Minor Arterials. In rural areas, minor arterials will provide a mix of interstate and intercounty travel service. In urban areas, minor arterials may carry local bus routes and provide intercounty connections, but they will not, for example, penetrate neighborhoods. When compared to the principal arterial system, the minor arterials provide lower travel speeds, accommodate shorter trips and distances and lower traffic volumes, but provide more access to property.

Chapter 17 “Freeways” and Chapter 16 “Rural and Urban Arterials” provide design guidance for freeways and arterials.

3.4.1.3 Collectors

Collector routes are characterized by a roughly even distribution of their access and mobility functions. Traffic volumes and speeds will typically be somewhat lower than those of arterials.

The function of rural collector roads is to serve intracounty travel needs and collect traffic flow from the rural local roads to the rural arterials and to distribute traffic flow from arterials back to the local roads. In rural areas, the collectors provide the following functions:

- provide access to adjacent land uses;
- carry traffic into areas with sparse development;

- serve larger towns and significant traffic generators (e.g., shipping ports, mining areas) that are not served by an arterial or freeway;
- spaced at intervals consistent with the traffic population density to accumulate traffic from local roads;
- provide service to smaller communities; and
- link locally important traffic generators with higher classified routes.

In urban areas, collector streets serve as intermediate links between the arterial system and points of origin and destination. Urban collectors typically have the following characteristics:

- provide both access and traffic circulation within residential neighborhoods and commercial/industrial areas;
- may penetrate residential neighborhoods or commercial/industrial areas to collect and distribute trips to and from the arterial system;
- in the Central Business District (CBD), may include the streets that are not classified as arterials;
- in fully developed areas, spacing generally is approximately ½ mile between routes and, within the CBD, between 650 feet and ½ mile;
- may be an urban extension of rural collector roads; and
- often include local bus routes.

Chapter 15 “Collector Roads and Streets” provides design guidance for collectors.

3.4.1.4 Local Roads and Streets

All public roads and streets not classified as arterials or collectors have a local road or street classification. Local roads and streets are characterized by their many points of direct access to adjacent properties and their relatively minor value in accommodating mobility. Speeds and volumes are usually low and trip distances short. Through traffic is often deliberately discouraged. Chapter 14 “Local Roads and Streets” provides design guidance for local roads and streets.

3.4.2 Geographic Classifications

The functional classification system is divided into urban and rural categories. Urban areas are defined as those places within boundaries having a population of 5,000 or more. Urban areas are further subdivided into urbanized areas (population of 50,000 and over) and small urban areas (population between 5,000 and 50,000). Rural areas are those areas outside the boundaries of urban areas. For design purposes, the designer should use the population forecast for the design year.

In many cases, a road defined as rural will go through a built-up area where the population is too small to be considered urban. However, it still retains the features of an urban area (e.g., signalized intersections, curbed streets, houses and business near the roadway, on-street parking). SCDOT defines these areas as urbanized.

3.5 SPEED

3.5.1 Definitions

The following speed definitions are commonly used in highway design:

1. Design Speed. Design speed is a selected speed used to determine the various geometric design features of the roadway. A design speed is selected for each project that will establish criteria for several design elements including horizontal and vertical curvature, superelevation and sight distance. In general, the speed relates to the driver's comfort and is not the speed at which a vehicle will lose control. Section 3.5.2 discusses the selection of design speed in general. Chapter 14 "Local Roads and Streets," Chapter 15 "Collector Roads and Streets," Chapter 16 "Rural and Urban Arterials," Chapter 17 "Freeways" and Chapter 18 "3R Projects (Non-Freeway)" present specific design speed criteria for various conditions.

2. Low Speed. For geometric design purposes, low speed is defined as 45 miles per hour or less.

3. High Speed. For geometric design purposes, high speed is defined as greater than 45 miles per hour.

4. Free-Flow Speed. (1) The theoretical speed when the density and flow rate on a study segment are both zero. (2) The prevailing speed on freeways at flow rates between 0 and 1000 passenger cars per hour per lane.

5. Operating Speed. Operating speed is the highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe speed as determined by the design speed on a section-by-section basis.

6. 85th-Percentile Speed. A speed value that is exceeded by 15 percent of the vehicles in a traffic stream. The most common application of the value is its use as one of the factors, and usually the most important factor, for determining the posted, legal speed limit of a highway section. In most cases, field measurements for the 85th-percentile speed will be conducted during off-peak hours when drivers are free to select their desired speed.

7. Pace. Pace is the range of speeds, in 10 miles per hour increments, in which the highest number of observations is recorded.

8. Posted Speed Limit. The posted speed limit corresponds to the value shown on regulatory signs as specified and described in the *Manual on Uniform Traffic Control Devices*. The posted speed is typically based on traffic and engineering investigations where statutory requirements do not apply. The selection of a posted speed is based on many factors including, but not limited to, the 85th percentile speed, roadside development, curb and gutter section, crash data, highway functional class and median type.

   The selection of a posted speed limit is based on several factors:

- design speed used during project development (posted speed is generally 5 miles per hour less than the design speed);
- median type on multilane facilities;
- 85th-percentile speed and pace speed;
- safe speed for curves or hazardous locations within the zone;
- highway functional classification and type of area;
- road surface characteristics, shoulder condition, grade, alignment and sight distance;
- type and density of roadside development and cultural/roadside friction;
- use of curb and gutter;
- safety experience;
- need for traffic signal progression;
- parking practices; and
- pedestrian and bicycle activity.

3.5.2 Design Speed Selection

The selected design speed is based on the following:

1. Posted/Regulatory Speed Limit. For all projects, the selected design speed should equal or exceed the anticipated posted or regulatory speed limit of the completed facility. This requirement recognizes the relationship between likely operating speeds and highway design. It also recognizes that the posted speed limit creates a driver expectation of safe operating speed.

2. Functional Classification. In general, the higher class facilities are designed with a higher design speed than the lower class facilities.

3. Urban/Rural. Design speeds in rural areas are generally higher than those in urban areas. This is consistent with the typically fewer constraints in rural areas (e.g., less development).

4. Balance. The selected design speed should be a reasonable balance between topography, urban and rural character and the functional class of the highway. A highway in level terrain may justify a higher design speed than one in rolling terrain, and a highway in a rural setting may justify a higher design speed than one in an urban area.

5. Terrain. The flatter the terrain, the higher the selected design speed may be. Lower design speeds may be used to minimize the higher construction costs often associated with terrain that is more rugged.

6. Traffic Volumes. Traffic volumes may impact the selection of design speed. A highway carrying a large volume of traffic may justify a higher design speed than a less important facility in similar topography. However, a low design speed should not be automatically assumed for a low traffic volume road where the topography is such that drivers are likely to travel at high speeds. Drivers do not adjust their speeds to the importance of the highway, but to the physical limitations and traffic using the facility.

7. Driver Expectancy. The selected design speed should be consistent with driver expectancy. The designer should consider the following when selecting a design speed:

   - avoid major changes in the design speed throughout the project limits;
   - where necessary, provide transitional design speeds between sections adjacent to the project; and
   - consider the expected posted speed in the selection of the design speed.

8. Range. Design speeds typically range between 30 mph and 75 mph depending upon urban/rural location and functional classification. For design applications, the selected design speed is typically in a 10-mph increment up to 50 mph, although 5-mph increments are acceptable (i.e., 35 mph and 45 mph). Depending upon the project application, 5-mph increments are used for design speeds from 50 mph to 75 mph.

9. 85th Percentile Speed. The 85th percentile speed is considered the appropriate speed limit even for those sections of roadway that have an inferred design speed lower than the 85th percentile speed. Posting a roadway's speed limit based on its 85th percentile speed is considered good and typical engineering practice.

10. Setting Lower Speeds. Arbitrarily setting lower design speeds is neither effective nor good engineering practice.

11. Inferred Design Speed. The inferred design speed is the maximum speed for which all critical design-speed-related criteria are met at a particular location.

12. New/Reconstructed Roadways. New or reconstructed roadways (and roadway sections) should be designed to accommodate operating speeds consistent with the roadway's highest anticipated posted speed limit based on the roadway's initial or ultimate function.

For geometric design application, the relationship between these design elements and the selected design speed reflects general cost-effective considerations. The value of a transportation facility in carrying goods and people is judged by its convenience and economy, which are directly related to its speed.

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3.6 TRAFFIC VOLUME CONTROLS

3.6.1 Definitions

The following terms are commonly used in the discussion of traffic volumes in highway design:

1. Annual Average Daily Traffic (AADT). The total traffic volume passing a point or segment of a highway facility in both directions of travel for one year divided by the number of days in the year.

2. Average Daily Traffic (ADT). The total traffic volume passing a point or segment of a highway facility in both directions of travel during a time period greater than one day but less than one year divided by the number of days in that time period. Although not precisely correct, ADT is often used interchangeably with AADT.

3. Capacity. The maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a given time period under prevailing roadway, traffic, environmental and control conditions. The time period most often used for analysis is 15 minutes.

4. Delay. The additional travel time experienced by a driver, passenger or pedestrian.

5. Density. The number of vehicles on a roadway segment averaged over space, usually expressed as vehicles per mile or vehicles per mile per lane.

6. Design Hourly Volume (DHV). The one-hour vehicular volume in both directions of travel in the design year selected for highway design. Note that, for capacity analyses, the DHV is typically converted to an hourly flow rate based on the maximum 15-minute flow rate during the DHV.

7. Directional Design Hourly Volume (DDHV). The peak one-hour volume in one direction of travel during the DHV.

8. Directional Distribution (D). The distribution, by percent, of the traffic in each direction of travel during the DHV, ADT and/or AADT.

9. Flow Rate. The equivalent hourly rate at which vehicles, bicycles or persons pass a point on a lane, roadway or other traffic way; computed as the number of vehicles, bicycles or persons passing the point, divided by the time interval (usually less than 1 hour) in which they pass. Expressed as vehicles, bicycles or persons per hour.

10. Level of Service (LOS). A qualitative measure describing operational conditions within a traffic stream, based on service measures such as speed and travel time, freedom to maneuver, traffic interruptions, comfort and convenience. In the *Highway Capacity Manual*, the qualitative descriptions of each level of service (A through F) have been converted into quantitative measures for the capacity analysis for each highway element, including:

    - freeway mainline,
    - freeway mainline/ramp junctions,
    - freeway weaving areas,

- two-lane rural highways,
- multilane rural highways,
- urban and suburban streets,
- signalized intersections,
- two-way stop-controlled intersections,
- all-way stop-controlled intersections,
- roundabouts,
- interchange ramp terminals, and
- off-street pedestrian and bicycle facilities.

The *Highway Capacity Manual* also provides procedures for determining the level of service for transit, pedestrians and bicyclists. Chapter 14 "Local Roads and Streets," Chapter 15 "Collector Roads and Streets," Chapter 16 "Rural and Urban Arterials," Chapter 17 "Freeways" and Chapter 18 "3R Projects (Non-Freeway)" present guidelines for selecting the LOS for traffic analyses in road design.

11. K. The ratio of DHV to ADT. K will vary based on the hour selected for design and the characteristics of the specific highway facility.

12. Peak-Hour Factor (PHF). A ratio of the volume occurring during the maximum-volume hour to the maximum rate of flow during a given time period within the peak hour (typically, 15 minutes). PHF may be expressed as follows:

$$\text{PHF} = \frac{\text{Peak Hour Volume}}{4(\text{Peak 15 minute Volume})}$$

13. Service Flow Rate. The maximum hourly rate at which vehicles, bicycles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a given time period (usually 15 minutes) under prevailing roadway, traffic, environmental and control conditions, while maintaining a designated level of service; expressed as vehicles, bicycles or persons per hour or vehicles per hour per lane.

3.6.2 Design Year Selection

3.6.2.1 Roadway Design

The geometric design of a highway should be developed to accommodate expected traffic volumes during the design life of the facility. This involves projecting the traffic volumes to a selected future year. Recommended design years are presented in Figure 3.6-A. The design year will be at least 20 years from the date the PS&E package is completed. Projected traffic volumes on State highways are provided by the Planning Division.

Project Scope of Work	Typical
New Construction/Reconstruction	20 Years
3R Projects (Non-Freeway)	Current*
Spot Improvements	Current*

* *In general, current traffic volumes may be used. However, if a project will introduce a new geometric design element (e.g., relocation of a horizontal curve), design the element using a 10-year projection and preferably a 20-year projection.*

RECOMMENDED DESIGN YEAR SELECTION
Figure 3.6-A

3.6.2.2 Other Highway Elements

The following presents the recommended criteria for selection of a design year for highway elements other than road design:

1. Bridges. The structural life of a bridge may be 75 years or more. For new bridges, bridge replacement and bridge reconstruction, the clear roadway width of the bridge will be based on the 20-year traffic volume projection from the date the PS&E package is completed. In addition, the designer may, on selected projects, evaluate if the bridge design will reasonably accommodate structural expansion to meet the clear roadway width across the bridge based on a traffic volume projection beyond 20 years.

2. Underpasses. The design year used for the geometric design of underpasses will be determined on a case-by-case basis.

3. Right-of-Way/Grading. The designer may consider potential right-of-way needs for the anticipated long-term corridor growth for a year considerably beyond that used for roadway design, especially in large metropolitan areas. No specific design year is recommended for use. For example, when selecting an initial median width on a divided highway, the designer may evaluate the potential need for future expansion of the facility to add through travel lanes. Other examples include potential future interchanges, potential reconstruction of a two-lane, two-way facility to a multilane highway, and the use of flatter side slopes to provide more future options.

4. Drainage Design. Drainage appurtenances are designed to accommodate a flow rate based on a specific design year or frequency of occurrence. The selected design year or frequency will be based on the functional class of the facility, the ADT and the specific drainage appurtenance (e.g., culvert).

5. Pavement Design. The pavement structure is designed to withstand the vehicular loads during the design analysis period without falling below a selected pavement serviceability rating. The design life for pavements is typically 20 years.

3.6.3 Design Hourly Volume

3.6.3.1 Selection

For most geometric design elements that are determined by traffic volumes, the peaking characteristics are most significant. The highway facility should be able to accommodate the design hourly volume (adjusted for the peak-hour factor) at the selected level of service. This design hourly volume (DHV) will affect many design elements including the number of through travel lanes, lane and shoulder widths and intersection geometrics. The designer should also analyze the proposed design using the a.m. and p.m. DHVs separately. This could have an impact on the geometric design of the highway.

See the *Highway Capacity Manual* for a detailed discussion on selecting the DHV. Because the design of the project is significantly dependent upon the projected design hourly volumes, carefully examine these projections before using them for design purposes.

3.6.3.2 Factors Affecting DHV Determinations

The following factors will affect the DHV determination:

1. K Factor. The proportion of ADT occurring in the design hour is commonly referred to as the K factor. The K factor is expressed as a decimal and is lowest on facilities where fluctuation of peak hour volumes is the least. The highest K values generally occur on recreational routes that have high seasonal, daily and hourly variations. K factors generally range between 0.09 in urban areas and 0.10 in rural areas. For highway sections with high peak periods and relatively low off-peak flows, the K factor may exceed 0.10. Conversely, for highways that demonstrate consistent and heavy flows for many hours of the day, the K factor is likely to be lower than 0.09.

2. Directional Flow. The proportion of DHV traffic traveling in the predominant direction is known as the directional factor (D). This factor reflects the imbalance in directional flow often observed, for example, where traffic is traveling toward employment centers in the a.m. peak and returning home in the p.m. peak. When D is applied to the DHV, it is known as the directional design hour volume (DDHV).

3. Composition. Composition of traffic is normally expressed as the percentage of trucks present in the traffic stream during the design hour. For geometric design and capacity studies, truck traffic is usually converted to passenger car equivalents, because trucks occupy more space and exhibit restrictive operational characteristics. For the purposes of design, light delivery trucks, pickups, etc., operate similarly to passenger cars and are considered passenger cars. The *Highway Capacity Manual* presents the passenger car equivalents for large trucks, single-unit trucks with dual rear wheels, buses and recreational vehicles based on the type of facility.

If the K factor, directional distribution and traffic composition are not provided, review the *Highway Capacity Manual* to determine the default values for the facility type being designed. Example 3.6-1 illustrates the procedure for converting ADT to DHV and DDHV.

* * * * * * * * * *

Example 3.6-1

Given: Two-lane, rural arterial
ADT = 5,080
K = 14 percent
D = 60 percent

Problem: Determine the DHV and DDHV for the facility.

Solution: The conversion formulas for non-directional ADT to DHV and DDHV are as follows:

DHV = ADT x K
DDHV = ADT x K x D

Therefore:

DHV = 5,080 x 0.14 = 711 vehicles per hour
DDHV = 5,080 x 0.14 x 0.60 = 427 vehicles per hour (in the predominant direction)

* * * * * * * * * * *

3.6.4 Capacity Analyses

3.6.4.1 Level of Service

Level of service (LOS) describes a qualitative measure of operational conditions within a traffic stream as perceived by motorists. A designated LOS is described in terms of average travel speed, density, traffic interruptions, comfort, convenience and safety.

Because drivers will accept different driving operational conditions, including lower travel speeds on different facilities, it is not practical to establish one level of service for application to every type of highway. Therefore, several levels have been established for the various classes and types of highways. The values of speed and design hourly volume used in each case to identify a level of service are the lowest acceptable speed and highest obtainable volume for that specific level.

Chapter 14 “Local Roads and Streets,” Chapter 15 “Collector Roads and Streets,” Chapter 16 “Rural and Urban Arterials,” Chapter 17 “Freeways” and Chapter 18 “3R Projects (Non-Freeway)” present LOS criteria for each highway type.

3.6.4.2 Objective

The highway mainline, intersection or interchange should be designed to accommodate the selected design hourly volume (DHV) at the selected level of service (LOS). This may involve adjusting the various highway factors that affect capacity until a design is determined that will accommodate the DHV. The detailed calculations, factors and methodologies are presented in the *Highway Capacity Manual* (HCM).

The designer should note that, in reality, the service flow rate of the facility is calculated. Capacity assumes a LOS E; the service flow rate is the maximum volume of traffic that a proposed highway of given geometrics is able to serve without the degree of congestion falling below a selected LOS. This is almost always higher than LOS E.

The HCM has established measures of effectiveness (MOE) for the level-of-service definition for each highway element on various types of highway facilities. These are presented in Figure 3.6-B. For each MOE, the HCM will provide the analytical tools to calculate the numerical value. The designer should note that highway capacity MOEs may be segregated into two broad categories; (1) uninterrupted flow, or open highway conditions; and (2) interrupted flow, as at stop-controlled or signalized intersections. Uninterrupted flow occurs on highways where the influence of intersections and abutting property development is not significant, and the design volume of a facility can be determined by an hourly rate of flow.

System element	HCM chapter	Service measure(s)				Systems analysis measure
		Automobile	Pedestrian	Bicycle	Transit	
Freeway facility	10	Density	—	—	—	Speed
Basic freeway segment	11	Density	—	—	—	Speed
Freeway weaving segment	12	Density	—	—	—	Speed
Freeway merge and diverge segments	13	Density	—	—	—	Speed
Multilane road	14	Density	—	LOS Score	—	Speed
Two-lane road	15	Percent time spent following, speed	—	LOS Score	—	Speed
Urban street facility	16	Speed	LOS Score	LOS Score	LOS Score	Speed
Urban street segment	17	Speed	LOS Score	LOS Score	LOS Score	Speed
Signalized intersection	18	Delay	LOS Score	LOS Score	—	Delay
Two-way stop	19	Delay	Delay	—	—	Delay
All-way stop	20	Delay	—	—	—	Delay
Roundabout	21	Delay	—	—	—	Delay
Interchange ramp terminal	22	Delay	—	—	—	Delay
Off-street pedestrian-bicycle facility	23	—	Space events	LOS Score	—	Speed

MEASURES OF EFFECTIVENESS FOR LEVEL OF SERVICE
Figure 3.6-B

The following presents the simplified procedure for conducting a capacity analysis for the highway mainline:

1. Select the design year (Section 3.6.2).

2. Determine the DHV (Section 3.6.4).

3. Select the level of service (see Chapters 14 through 18).

4. Document the proposed highway geometric design (lane width, length of weaving section, number and width of approach lanes at intersections, etc.).

5. Using the *Highway Capacity Manual*, analyze the capacity of the highway element for the proposed design.

6. Compare the calculated measured level of service (LOS) with the desired LOS. If the calculated LOS is greater than or equal to the desired LOS, the proposed design will generally meet the objectives of the capacity analysis. If the LOS is lower than the desired LOS, the proposed design may need further evaluation. The designer either should adjust the highway design or adjust one of the capacity elements (e.g., the selected design year or the level-of-service goal).

The traffic designer will perform and review the results of the capacity analyses. Software used for the analyses must be approved by the Department.

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3.7 DESIGN VEHICLES

The physical and operational characteristics of vehicles using the highway are important controls in roadway design. Design criteria may vary according to the type of vehicle and the volume of each type of vehicle in the traffic stream.

Vehicular characteristics that impact design include:

1. Size. Vehicular sizes determine lane and shoulder widths, vertical clearances and, indirectly, highway capacity calculations.

2. Offtracking. The design of intersection turning radii, traveled way widening for horizontal curves and pavement widths for interchange ramps are usually controlled by the largest design vehicle likely to use the facility with some frequency.

3. Storage Requirements. Auxiliary lane storage lengths, bus turnouts and parking lot layouts are determined by the number and types of vehicles to be accommodated.

4. Sight Distance. Eye height and braking distances vary for passenger cars and trucks, which can impact sight distance considerations.

5. Acceleration and Deceleration. Acceleration and deceleration rates often govern the dimensioning of such design features as speed-change lanes at intersections and interchange ramps and climbing lanes.

6. Vehicular Stability. Certain vehicles with high centers of gravity may be prone to skidding or overturning, affecting design speed selection and cross slope design elements.

The following design vehicles are used for highway design in South Carolina:

1. Passenger Car (P). Passenger cars are used in the design for most geometric design criteria (e.g., stopping sight distance, intersection sight distance, horizontal alignment, cross section widths, acceleration and deceleration lanes).

2. Large School Bus (S-Bus-40). This vehicle is the minimum design vehicle that should be used for intersection and entrance/exit designs, including auxiliary turn lanes where bus traffic is present.

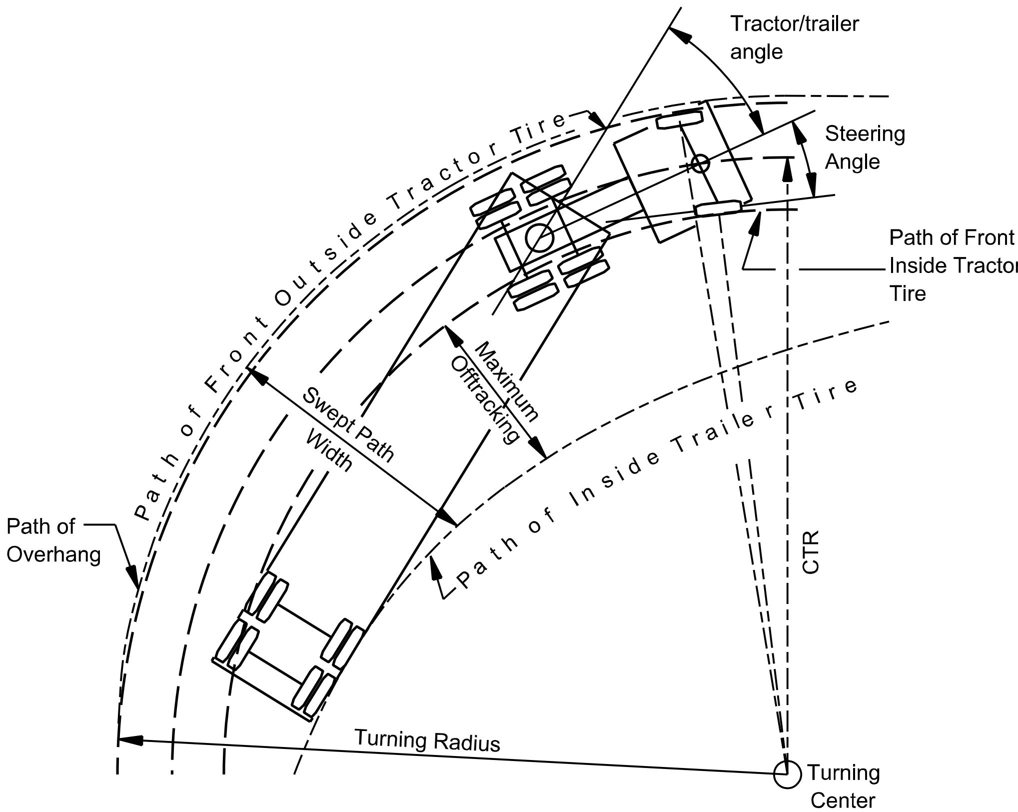
3. Interstate Semitrailer (WB-62). This vehicle is the minimum design vehicle that should be used for intersection and entrance/exit designs, including auxiliary turn lanes where truck traffic is present.

4. Interstate Semitrailer (WB-67). This vehicle is the minimum design vehicle that should be used for design of storage/clearance for through and auxiliary turn lanes where truck traffic is present.

5. Motor Home and Boat Trailer (MH/B). This vehicle should be used for designs of auxiliary turn lanes at intersections and entrances/exits where motor homes are present.

Figure 3.7-A presents turning characteristics of a typical tractor-semitrailer combination truck. Information on the vehicular dimensions and minimum turning radii for the above mentioned design vehicles are provided in the AASHTO *A Policy on Geometric Design of Highways and Streets.* Computer-simulated, turning templates are available for these design vehicles (e.g., AutoTurn).

The selection of appropriate design vehicles for intersections and interchanges is discussed in Chapter 9 “Intersections” and Chapter 10 “Interchanges,” respectively. In general, the designer should select the largest design vehicle that will use the facility with some frequency. However, the designer should consider local restrictions and the occasional larger vehicle that may use the facility.



Definitions:

1. Turning Radius. The circular arc formed by the turning path radius of the front outside tire of a vehicle. Vehicular manufacturers also describe this radius as the turning curb radius.

2. CTR. The turning radius assumed by a designer when investigating turning paths. It is set at the center of the front axle of a vehicle.

3. Offtracking. The difference in the paths of the front and rear wheels of a vehicle as it negotiates a turn. The path of each rearward tire of a turning vehicle does not coincide with that of the corresponding forward tire. This phenomenon is shown in the drawing above.

4. Swept Path Width. The amount of roadway width that a vehicle covers in negotiating a turn equal to the amount of offtracking plus the width of the vehicle. The most significant dimension affecting the swept path width of a tractor/semitrailer is the distance from the kingpin to the rear trailer axle or axles. The greater this distance, the greater the swept path width.

5. Steering Angle. The maximum angle of turn built into the steering mechanism of the front wheels of a vehicle. This maximum angle controls the minimum turning radius of the vehicle.

6. Tractor/Trailer Angle. The angle between adjoining units of a tractor/semitrailer when the combination unit is placed into a turn. This angle is measured between the longitudinal axes of the tractor and trailer as the vehicle turns. The maximum tractor/trailer angle occurs when a vehicle makes a 180 degree turn at the minimum turning radius and is reached slightly beyond the point where a maximum swept path width is achieved.

TURNING CHARACTERISTICS OF A TYPICAL DESIGN VEHICLE
Figure 3.7-A

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3.8 ACCESS MANAGEMENT

3.8.1 Definitions

The following definitions apply to access management and access control:

1. Access Management. The process of governing access to land development by a public agency where the agency considers the highway facility and its surrounding activities as part of an overall system. Individual parts of the system (e.g., zoning, land-use planning, site plan development, driveway permits, public transportation, roadway network) should be properly integrated and coordinated. Through proper application of access management, the objectives of providing safe and efficient traffic flow coupled with access to abutting properties can be achieved.

2. Control by Regulation. All highways warrant access management by permit or by design. Control by regulation is exercised by the Department, county highway departments or municipalities to specify the location of private accesses to and from the public road system. Occasionally, statutory control is used to restrict access to only public roads and major traffic generators. Zoning may be used to effectively control development on adjacent property so that major generators do not hinder traffic operations. However, zoning restrictions are at the discretion of the local government. Driveway regulations and permits are used to control the geometric design of an entrance, driveway spacing and driveway proximity to public road intersections.

3. Access Control. The condition where the public authority regulates the right of abutting owners to have access to and from a public highway by declaring the highway to be either fully or partially access controlled. This is accomplished through the purchase of access rights, driveway controls, turning restrictions or geometric design (e.g., grade separations).

   South Carolina Code 57-5-1010 states: "*Controlled-access facility means a State highway or section of State highway especially designed for through traffic, and over, from or to which highway owners or occupants of abutting property or others shall have only a controlled right or easement of access."* The abutting landowner or others have no legal right of access except at points and manners as determined by the Department.

   a. Full Control of Access/Limited Access. Highways that are designated to have full control of access are referred to as freeways. Priority is given to through traffic and access to the highway is only provided at interchanges with selected public roads. All other intersecting roads are terminated at the right of way line, perpetuated with grade separations or interconnected with other roads. Access is provided to properties abutting the freeway via frontage roads, service drives or the existing public road system. Full control of access maximizes the capacity, safety and vehicular speeds on the highway.

   b. Partial Control of Access/Limited Access. An expressway design is the common term used for this type of facility. Priority is given to through traffic. Some intersections will be provided and private entrance connections will be allowed by permit. The proper selection and spacing of intersections and other connections provides a balance between the mobility and access functions of the highway.

4. Control of Access Line. A line established by the Department that delineates areas where ingress to and egress from a highway facility is controlled by Purchase of Access Rights. When an existing access-controlled highway is reconstructed, the access control lines should be reviewed for possible revisions.

3.8.2 Access Management

Access management is a public authority's selective use of regulations, policies and procedures to limit or control public access to and from property abutting highways. It plays an important part in providing a safe and efficient highway system.

Highway transportation involves the movement of persons and goods along highways. A complete system of transportation services must also provide access to abutting properties. There is a trade-off between the mobility and access uses of a highway. In developed areas, arterial highways are particularly susceptible to decisions that conflict with and compromise the principal function of mobility for which the arterial was designed. Generally, the high-capacity, full-controlled access highway and the local access residential or commercial street, at the other end of the capacity range, present the fewest access management problems and best meet service characteristics noted in the functional classification system.

Traditionally, as a community grows, land subdivides and develops, and businesses are attracted to busy highways. Direct and frequent access is obtained by constructing driveways and intersections. To be effective, access management should be addressed in the early stages of land development. As crossroad traffic volumes increase, traffic signals are warranted, and the need to plan for major access points is critical to the mobility function of major highways. Total disregard of access management can create traffic congestion and lead to public demands for better transportation service.

The control and regulation of access to roadways offers the following benefits:

- protects the safety of the motoring public,
- protects the level of service and carrying capacity of the roadway,
- provides reasonable access to development in accordance with needs, and
- delays the need for expensive new projects.

The designer should consider the following objectives of access management:

- limit the number of conflict points,
- separate conflict areas,
- limit the severity of conflicts,
- limit vehicular speed change requirements, and
- maintain the performance of the roadway.

3.8.3 Type of Control

The Project Planning Report should identify the type of control proposed for each project. Full control of access will be provided on all freeways. Additionally, full control of access may be used on other routes of importance as approved on a case-by-case basis. Other routes may be designed using limited control of access. Where it is necessary to revise the control access line, the designer should coordinate with the Rights of Way Office.

Clearly denote Control of Access or Limited Access lines on the plan sheets; see Chapter 22 "Plan Sheets Preparation."

3.8.4 Criteria

3.8.4.1 Driveways

Spacing driveways apart from each other can reduce the number and severity of crashes. See the *SCDOT Access and Roadside Management Standards* (ARMS) for the minimum driveway spacing requirements. The driveway spacing is measured from the near edge to near edge of the adjacent driveways. See the *SCDOT ARMS* for additional guidance on access control at driveways.

3.8.4.2 Intersections

Locate all points of access as far from the roadway intersections or railroads as feasible and practical. See the *SCDOT ARMS* for guidance. In determining the access lines, also consider the following:

1. Intersection Radii. Do not locate an access within the radius of the intersecting roadways.

2. Triangular Right-of-Way Area. At intersections where the Department has purchased the triangular or sight distance areas, driveways are not permitted to cross or enter the area, except where the elongation of areas may warrant special considerations.

3. Limited Access Facilities. Projects that allow vehicular access to the mainline, via at-grade intersections, are considered Limited Access. The control access line will turn away from the mainline facility and follow the side road right of way for a distance shown in the *SCDOT ARMS*. For right-in, right-out access, use a minimum of 150 feet or the value given in the *SCDOT ARMS* if it is less than 150 feet.

4. Median Openings. A median crossover may be permitted when an engineering review indicates that all of the following conditions are met:

    - The spacing to the nearest crossover is at least 500 feet in urban areas and 1000 feet in rural areas (centerline to centerline).
    - Where necessary, provide a left-turn lane and taper as discussed in Chapter 9 "Intersections."
    - The sight distance criteria from Chapter 4 "Sight Distance" are met.
    - Significant traffic volumes will be generated.

- The operation of the highway, other accesses or crossovers will not be adversely affected.

- The maximum grade on the crossover does not exceed 5 percent.

5. Unsignalized Intersection Spacing. If practical, avoid short distances between intersections because they tend to impede traffic operations. Where practical, realign the roadways to form a single intersection.

   To operate efficiently, urban intersections should be a minimum of 500 feet apart. For rural areas, provide a minimum spacing of ¼ mile and, desirably, ½ mile apart. Generally, treat signalized and unsignalized intersections the same. Because of changing traffic patterns, development and crash concerns, unsignalized intersections may be converted to signalized intersections in the future. Conduct a traffic analysis to determine if free-flow can be obtained between the intersections.

   In addition, avoid short gaps between opposing T intersections. Drivers tend to encroach into the opposing lanes (corner cutting) so that they can make their turning maneuvers in one movement. In general, all new intersections should preferably be at least 500 to 700 feet apart. However, intersections with an offset to the right should have adequate spacing to properly develop left-turn lanes.

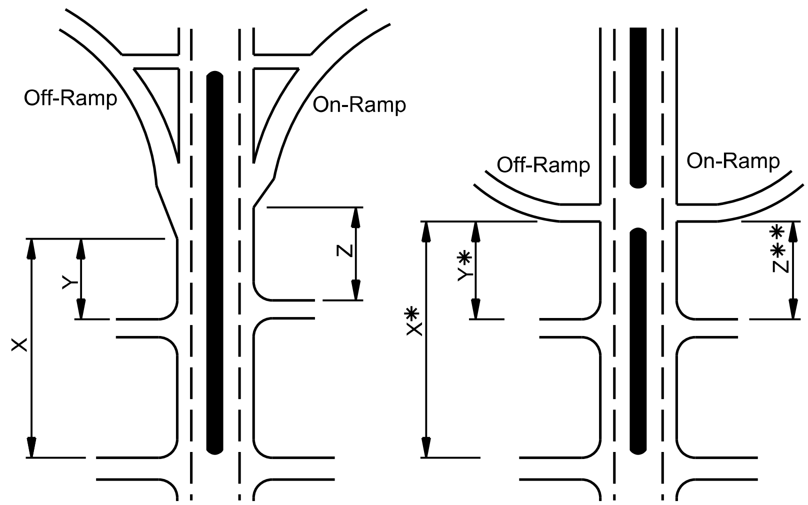
6. Signalized Intersection Spacing. Signalized intersections that are too close to each other may result in unnecessary delay, frequent vehicle stops/starts and increased fuel consumption and emissions. Correctly spaced signalized intersections will allow traffic signal timing plans to efficiently accommodate all types of traffic conditions. The minimum spacing requirements for signalized intersections are shown in Figure 3.8-A. If there is no reasonable alternative and a queue analysis can be conducted that shows adequate spacing, then the minimum traffic signal spacing requirements may be less.

Functional Class	Traffic Signal Spacing (ft)
Major Arterial	2640
Minor Arterial	1320
Collector	1320
Local	1320

MINIMUM TRAFFIC SIGNAL SPACING
Figure 3.8-A

3.8.4.3 Interchanges

Proper access control must be provided along the crossing road near the ramp/crossing road intersection or along a frontage road where present. This will ensure that the intersection has approximately the same degree of freedom and absence of conflict as the freeway itself. The access control criteria should be consistent with these goals. For new construction, the minimum spacing requirements are shown in Figure 3.8-B.



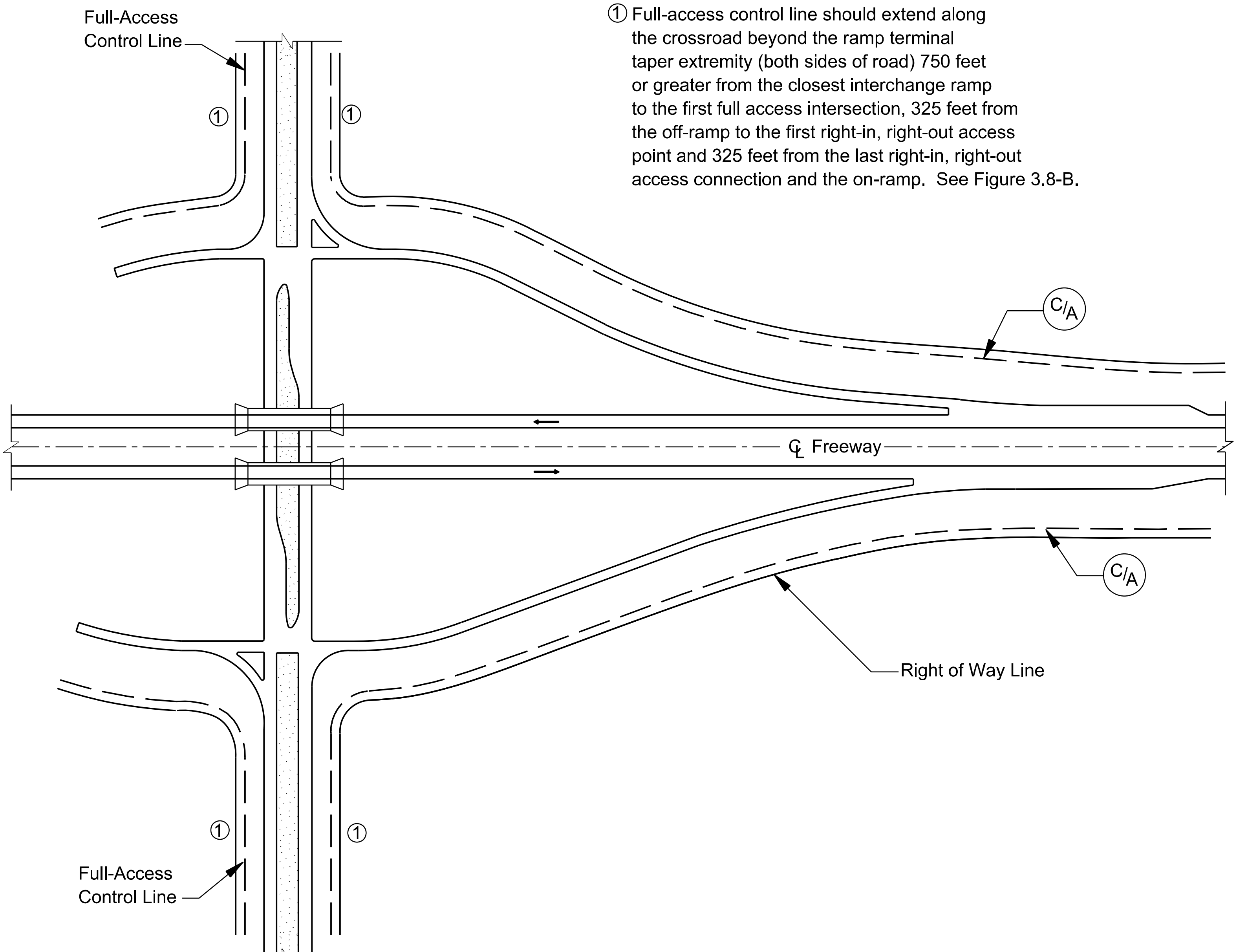
✱ Measured from the end of the lane taper if an acceleration lane exists for the off-ramp.
✱✱ Measured from the beginning of the lane tape if a turn lane for the on-ramp exists.

	Distance (ft)	Description
X	750	Distance from the closest interchange ramp to the first full access intersection
Y	325	Distance from the off-ramp to the first right in, right out access point
Z	325	Distance between the last right in, right out access connection and the on-ramp

MINIMUM SPACING FOR FREEWAY INTERCHANGES
Figure 3.8-B

Figures 3.8-C and 3.8-D present access control for several typical interchange designs. These provide SCDOT guidance for the location of the limits of control of access lines along the ramp, at ramp/crossroad intersections, across from the ramp terminal and along frontage roads.

As indicated in the figures, the control of access limits extends a distance along the crossing road away from the ramp or frontage road intersection. In areas where the potential for development exists that would create traffic problems, it may be appropriate to consider minimum lengths of access control. In addition, many areas have changed over the years from rural to urban. A change in area character alone is not a sufficient justification to alter the location of the control of access line when an existing interchange will be rehabilitated or when SCDOT receives requests for additional access points from outside interests.

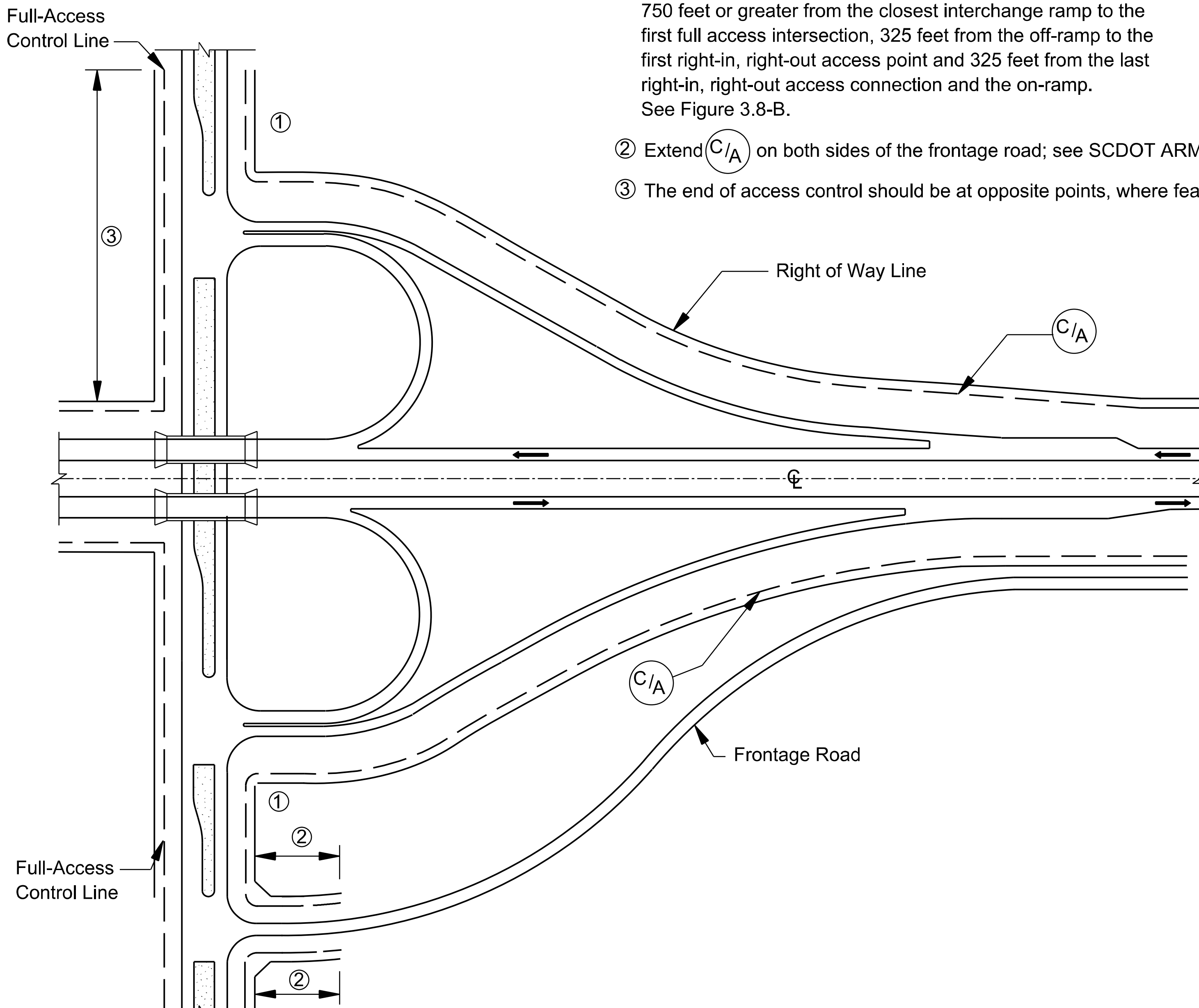


TYPICAL ACCESS CONTROL FOR A DIAMOND INTERCHANGE
Figure 3.8-C

① Full-access control line should extend along the crossroad beyond the ramp terminal taper extremity (both sides of road) 750 feet or greater from the closest interchange ramp to the first full access intersection, 325 feet from the off-ramp to the first right-in, right-out access point and 325 feet from the last right-in, right-out access connection and the on-ramp. See Figure 3.8-B.

② Extend C/A on both sides of the frontage road; see SCDOT ARMS.

③ The end of access control should be at opposite points, where feasible.



TYPICAL ACCESS CONTROL FOR A PARTIAL CLOVERLEAF INTERCHANGE
Figure 3.8-D

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3.9 ENVIRONMENTAL PROCEDURES

The Environmental Services Office is responsible for a variety of activities related to environmental impacts and procedures. This includes air, noise and water quality analyses; social, economic, biological, archeological and historical impacts; preparation of environmental documents for SCDOT projects; evaluation and mitigation of hazardous waste sites; and the public's involvement with the environmental document. The Environmental Services Office coordinates with the applicable Federal or State agencies to process the environmental documentation and permit applications.

It is important for the road designer to understand the various environmental activities and procedures to ensure that the proper environmental documentation and permits are completed as part of the project development. The entire Project Development Team is responsible for calling attention to potential environmental issues throughout the development of the project. The need to receive one or more permits or approvals can significantly affect the project schedule.

See the *SCDOT Environmental Reference Document* for information on the Department's criteria for conducting environmental analyses and preparing environmental documents.

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3.10 ROADWAY DRAINAGE

Proper roadway drainage is a critical element in protecting the structural integrity of the highway system, ensuring the safety of the traveling public and avoiding adverse impacts on adjacent property. A significant portion of highway construction funds are expended on drainage-related items.

3.10.1 Resources

The designer should review the following resources for roadway drainage design:

1. The *SCDOT Requirements for Hydraulic Design Studies* provides a comprehensive discussion on the Department's hydrologic and hydraulic practices. This publication can be found on the Department's internet site.

2. The Hydraulic Engineering webpage on the Department's internet also provides several other resources for the roadway drainage design (e.g., rainfall intensity values, inlet spacing charts, GeoPAK guidance, plan sheet templates).

3. The *SCDOT Water Quality Manual* is a guidance document developed to provide engineers, plan reviewers, inspectors and contractors information on the stormwater quality management requirements for SCDOT. This publication can be found on the Department's internet site.

4. Chapter 21 "Procedures for Highway Plans Preparation" and Chapter 22 "Plan Sheets Preparation" of this *Manual* provide guidance for incorporating the hydraulic design criteria on the contract plans.

5. Instructional Bulletin 2010-1 "Culvert Pipe Structural Design Criteria" provides design guidance for the fill height tables provided in the *SCDOT Standard Drawings* for each pipe material.

6. Engineering Directive 23 "Standards for Stormwater Management and Sediment Reduction" describes the Department's criteria for stormwater management and water quality.

7. Engineering Directive 24 "Selection of Drainage Pipe for use in South Carolina" describes the Department's procedures for selection of allowable culvert types 12-inch or larger for State highways.

8. Engineering Directive 26 "Dams/Water Impoundments Adjacent to Highways" describes the Department's procedures for evaluating dams and other water impoundment devices adjacent to State projects to ensure these devices are safe.

9. Engineering Directive 27 "Drainage Outfalls" describes the Department's procedures for evaluating changes to drainage channels and pipes and their effect on nearby property owners.

3.10.2 Definitions

The following presents selected definitions that have an application to roadway drainage:

1. 100-Year Flood. A flood volume (or discharge) level that has a 1 percent chance of being equaled or exceeded in any given year.
2. 500-Year Flood. A flood volume (or discharge) level that has a 0.2 percent chance of being equaled or exceeded in any given year.
3. Allowable Headwater Depth. The depth or elevation of the flow impoundment for a drainage facility (e.g., a culvert) above which damage or a significant flood hazard could occur.
4. Backwater. The increase in water surface elevation relative to the elevation occurring under natural channel and floodplain conditions (upstream of a highway facility).
5. Base Flood. The design flood used for hydrologic calculations that are based on the roadway classification.
6. Base Floodplain. The area subject to flooding by the base flood.
7. Channel Change. A modification to the natural alignment of a channel (stream) necessitated by highway construction. Channel changes should only be used where absolutely essential (e.g., where the natural channel will be covered in fill).
8. Culvert. A structure that is a) usually a closed conduit designed hydraulically to take advantage of submergence to increase hydraulic capacity, and b) used to convey surface runoff through a highway or railroad embankment. AASHTO classifies a culvert as a structure of less than a 20-foot span as measured along the roadway centerline. A culvert is a structure, as distinguished from a bridge, that is usually covered with embankment and is composed of structural material around the entire perimeter.
9. Design Flood Frequency. The flood frequency selected for determining the necessary size of the drainage appurtenance.
10. Detention Pond. A basin, pond or reservoir incorporated into the watershed where runoff is temporarily stored, thus attenuating the peak of the runoff hydrograph. A stormwater management facility that temporarily impounds runoff and discharges it through a hydraulic outlet structure to a downstream conveyance system.
11. Ditch Check Dam. A small temporary dam constructed across a swale or drainage ditch that acts as a filter trapping soil particles and allowing water to flow through. See *SCDOT Standard Drawings*.
12. Drainage Area. The catchment area for rainfall and other forms of precipitation that is delineated as the watershed producing runoff (i.e., the contributing watershed).
13. Drainage End Treatment. A structure, commonly made of concrete or metal, that is attached to the end of a culvert or pipe for such purposes as retaining the embankment from spilling into the waterway, improving the appearance, providing anchorage,

improving the culvert efficiency, limiting some scour at the outlet and/or improving roadside safety.

14. Erosion Control. Mitigation measures used to reduce (or decelerate) erosion, which is a natural or geologic process whereby soil materials are detached and transported from one location and deposited elsewhere, primarily due to rainfall, runoff and wind.

15. Flood Frequency. The number of times a flood of a given magnitude can be expected to occur on average over a specified period of time.

16. Floodplain. Any plain that borders a stream and is covered by its waters in time of flood. A nearly flat, alluvial lowland bordering a stream and commonly formed by stream processes that is subject to inundation by floods.

17. Headwater Depth. Depth of water above the inlet flow line at the entrance of a culvert or similar structure. Depth of water upstream of a contraction such as occurs at a bridge or similar structure. Natural flow depth plus backwater caused by a drainage structure.

18. Hydraulics. The applied science concerned with the behavior and flow of liquids, especially in pipes, channels, structures and the ground. In highway drainage, the science addressing the characteristics of fluid mechanics involved with the flow of water in or through drainage facilities.

19. Hydrology. The science that explores the interrelationship between water on the earth and in the atmosphere. In hydraulic practice for highways, hydrology is used to calculate discharges for a given site based on the site characteristics.

20. Intensity. The rate of rainfall upon a watershed, usually expressed in inches per hour.

21. Maximum Allowable Backwater. The maximum amount of backwater that is acceptable to the Department for a proposed facility based on State and Federal laws and on Department policies.

22. Outfall Ditch. A channel that directs stormwater discharge from a roadway facility.

23. Peak Discharge (Peak Flow). The maximum rate of flow passing a given point during or after a rainfall event or snow melt. For example, the peak discharge for a 100-year flood is expressed as Q_{100}.

24. Recurrence Interval (Return Period). The average number of years between occurrences of a discharge of a particular magnitude. For example, the recurrence interval for a 100-year flood discharge is 100 years.

25. Regulated Floodway. The floodplain area that is reserved in an open manner by Federal, State or local requirements (i.e., unconfined or unobstructed either horizontally or vertically) to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program (NFIP).

26. Retention Pond. A basin, pond or reservoir where water is stored for regulating a flood. The stored runoff is disposed of by such means as infiltration, by injection (or dry) wells or by release to the downstream drainage system after the storm event. The release may be through a gate-controlled gravity system, pumping or an outlet structure.

27. Routed Flow. The process whereby a peak flow and/or its associated streamflow hydrograph is mathematically transposed to another site downstream considering the effect of channel storage.

28. Runoff Coefficient. A factor, dependent on terrain and topography, representing that portion of runoff that results from a unit of rainfall. More simply stated, the rate of runoff to precipitation.

29. Silt (or Sedimentation) Basin. A storage area, either temporary or permanent, used to detain sediment-laden runoff from disturbed areas for a sufficient length of time for the majority of the sediment to settle.

30. Spread. The transverse encroachment of stormwater onto a street where flow has accumulated in and next to the roadway gutter. This water may represent an interruption to traffic flow, splash-related problems and a source of hydroplaning during rainstorms.

31. Trench Drain. A long, narrow drainage inlet that extends for a considerable length to intercept flow before exiting onto a roadway or to collect gutter flow to reduce ponding depth and spread at curb inlets. See Section 3.10.3.8.

32. Velocity. The rate of travel of a stream or river or of the objects or particles transported therein, usually expressed in distance per unit time.

3.10.3 Roadway Drainage Criteria

The *SCDOT Requirements for Hydraulic Design Studies* documents the Department's hydrologic and hydraulic criteria for the design of roadway drainage appurtenances. This Section presents a few details of special interest to the road designer.

3.10.3.1 General

Most highway projects require new drainage facilities and/or the improvement of existing drainage systems. This may be in the form of earth or lined, channels, streams, culverts, closed drainage systems, etc. A specific project may incorporate any or all of these drainage requirements. The designer must be knowledgeable of Department drainage policies and practices affecting road design elements.

3.10.3.2 Inlet Spacing

The designer is required to show drainage inlets on the preliminary plans. The final placement of the inlets will be based on the hydraulic design study. Minimum inlet spacing is 150 feet with a maximum spacing of 400 feet. Inlets are required at locations needed to collect runoff to meet the design controls specified by the Department's design criteria (e.g., allowable water spread, design year). In addition, there are many locations where inlets may be necessary without regard to contributing drainage area. These locations should be marked on the plans prior to any computations of discharge, water spread, inlet capacity or bypass. Examples of these locations are as follows:

1. Place inlets at low points (e.g., sags) and at intersections as required to intercept the flow.

2. Unless a hydraulic analysis indicates otherwise, base inlet spacing on the inlet spacing charts provided on the Department's internet site.

3. Place inlets upstream of median breaks, entrance/exit ramp gores, crosswalks and cross slope transitions.

4. Place inlets immediately before and after bridges.

5. Re-space inlets following the field review, if required.

6. Other methods may be used to supplement catch basins and expand the efficiency and capacity of drainage (e.g., trench drains or extended throats).

3.10.3.3 Sideline Pipes

Sideline pipes are identified as longitudinal pipe culverts in roadway ditches at driveways and other locations. The policy for establishing pipe lengths for standard driveways is as follows:

1. Additional pipe (of various sizes) may be shown in the plans as an inclusion item (to be determined during the Design Field Review).

2. If additional (or less) pipe length is required at driveways during construction, the Resident Construction Engineer will make these determinations.

3.10.3.4 Crossline Pipes

In calculating the length of pipe required to span the fill wherein beveled or flared end sections will be used, give consideration to the usable length of a beveled or flared end section.

For the use of pipe end structures with respect to roadside safety, see AASHTO *Roadside Design Guide*.

3.10.3.5 Paved Gutters

Use Figure 3.10-A to determine the limits of paved gutters.

Gutter Design Grade	Distance to Carry Surface Runoff Before Beginning Paved Gutter
0 to 0.49%	None Required
0.50% to 0.99%	1,000 linear feet
1.00% to 1.99%	500 linear feet
2.00% or greater	250 linear feet

ASPHALT GUTTER
Figure 3.10-A

3.10.3.6 Box Culvert Extensions

Desirably, provide a space of 10 feet between the wingwall ends of box culvert extensions and the present or new right of way. Acquire additional right of way as required at each site to encompass permanent erosion control devices (e.g., energy dissipators, paved liners) placed at the ends of box culvert extensions.

In some cases, the designer may be responsible for designing large junction boxes for box culverts.

3.10.3.7 Bridge Drainage

For all projects, Department policy is to include bridge end drainage, which is designed to intercept and capture all gutter flow as near as practical to each end of the bridge.

Good drainage design at the ends of bridges is essential for proper drainage. At bridge ends where the approach roadway does not have curb and gutter, the typical Department practice is to use an asphalt flume if the asphalt flume is appropriate. If the asphalt flume is not appropriate, an alternative design must be used.

In situations where an asphalt flume is not appropriate, bridge end drainage may be designed using catch basins, grate inlets, curb opening inlets or combination inlets. The hydraulic characteristics of the inlets should be considered in selecting the type. The designer must provide appropriate details that properly control and direct the flow of water to the bridge end drainage structure. The designer must also provide details that address how the bridge end drainage components interface with all other impacted roadside design features.

Inlets on the bridge should be spaced to minimize runoff entering the bridge approaches. Design collectors at the downslope end of the bridge to collect all of the flow not intercepted by the bridge deck inlets. If there are no bridge deck inlets, provide downslope inlets to intercept all of the bridge drainage. Provide a pipe, paved channel or trough to transport the water down the surface of the embankment.

At bridge ends where the approach roadway has curb and gutter, catch basins should be detailed as close as possible to the approach slabs. See the *SCDOT Standard Drawings* for additional information on bridge end drainage.

Section 6.2.3.3 discusses vertical alignment requirements for open drainage on bridges.

3.10.3.8 Trench Drains

Consider trench drains where surface flows are suspected to interfere with traffic operations. Runoff from an adjacent property through a driveway toward the roadway can be intercepted by a trench drain installed across the driveway and conveyed into the parallel ditch or into a drainage box.

In curb-and-gutter sections, the typical section provides for runoff to reach the gutter. However, when rehabilitating and widening a section of roadway that was previously a ditch section, but is now being designed as a curb-and-gutter section, grades, vertical curves and superelevation rotation can prohibit conveying the runoff to the desired catch basins and storm sewers. Typically, the minimum desired gutter grade is 0.5 percent; however, 0.3 percent may be used with adequate cross slope. The length of curve can create relatively flat locations on a crest and in a sag vertical curve. Where feasible, catch basin spacing may be reduced to facilitate the efficiency of the drainage system.

Where additional pipe and catch basins are not feasible or the area is not conducive to a catch basin (e.g., in a driveway), then trench drains may be installed in the gutters to enhance the roadway drainage. Trench drains in gutters will reduce potential ponding in the gutter area caused by inherent, nearly flat grades occurring in pavement transitions and in the low and high points of vertical curves. Typically, the flow line of a trench drain is fixed at 0.6 percent, but will vary according to the grade of the gutter. Trench drains can be placed in an opposing direction to the gutter grade, if the gutter grade does not exceed 0.2 percent in the opposite direction. For example, this would yield a trench drain flow line grade of 0.4 percent in a gutter with an opposing grade of 0.2 percent. This composite grade of the trench drain flow line should not be less than 0.4 percent.

Consider the following guidelines where trench drains are used to supplement drainage in gutters:

- Where calculated individual longitudinal grades in the gutter are ≤ 0.1 percent, check the actual elevations on profile to determine percent grade in vertical curves.
- Provide a drainage box within 96 linear feet to outlet the trench drain.
- Design the trench drain in 16-foot increments. The maximum length of trench drain in one run is 96 linear feet.
- Place the location and quantity information on the “General Construction Note” Sheet.

See *SCDOT Standard Drawings* for details of trench drain placement, measurement and payment.

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3.11 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. *Highway Capacity Manual 2010*, Transportation Research Board, 2010.

3. *Access and Roadside Management Standards*, SCDOT, 2008.

4. *Mitigation Strategies for Design Exceptions*, FHWA, 2007.

5. Environmental Reference Document, SCDOT, 2008.

6. Requirements for Hydraulic Design Studies, SCDOT, 2009.

7. Memorandum of Agreement for Federal-Aid Preventive Maintenance Projects, May 2015.

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Chapter 4

SIGHT DISTANCE

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 4
SIGHT DISTANCE

Sight distance is the length of the roadway ahead that is visible to the driver. This chapter discusses stopping, passing, decision and intersection sight distances.

4.1 STOPPING SIGHT DISTANCE

The available sight distance on a roadway should be long enough to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater lengths of visible roadway are desirable, the sight distance at every point along a roadway should be at least that needed for a below average driver or a vehicle to stop. Stopping sight distance (SSD) is the sum of the distance traveled during a driver's perception/reaction or brake reaction time and the distance traveled while braking to a stop.

4.1.1 Assumptions

The AASHTO *A Policy on Geometric Design of Highways and Streets* presents the basic equations for determining SSD. The following briefly discusses the basic assumptions within the SSD model:

1. Brake Reaction Time. This is the time interval between when the obstacle in the road can be physically seen and when the driver first applies the brakes. Based on several studies of observed driver reactions, the assumed value is 2.5 seconds. This time is considered adequate for approximately 90 percent of drivers in simple to moderately complex highway environments.

2. Braking Action. The braking action is based on the driver's ability to decelerate the vehicle while staying within the travel lane and maintaining steering control during the braking maneuver. A deceleration rate of 11.2 feet/second/second is considered comfortable for 90 percent of the drivers.

3. Speed. The highway's design speed is used to determine the initial driver speed.

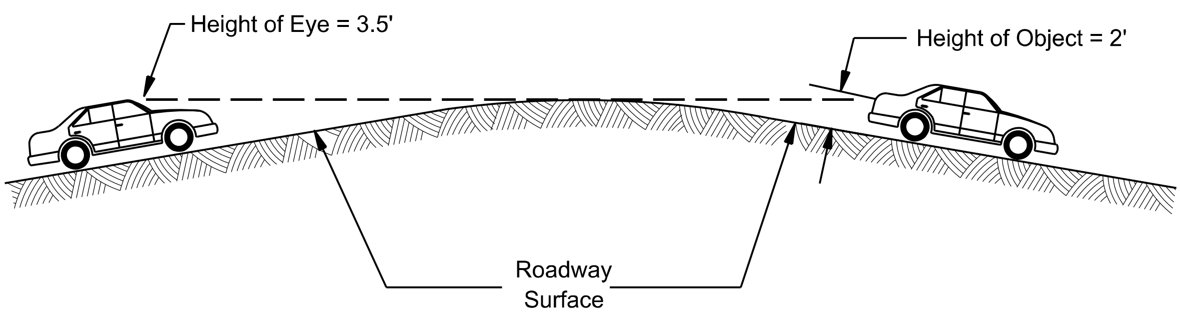
4.1.2 Level Grade

Figure 4.1-A provides stopping sight distances for passenger cars on grades less than 3 percent. Use values that meet or exceed the required stopping distance for design. When applying the SSD values, the height of eye is assumed to be 3.5 feet and the height of object 2 feet. Figure 4.1-B provides a graphical representation of SSD criteria.

Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance On Level Grade (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
15	55.1	21.6	76.7	80
20	73.5	38.4	111.9	115
25	91.9	60.0	151.9	155
30	110.3	86.4	196.7	200
35	128.6	117.6	246.2	250
40	147.0	153.6	300.6	305
45	165.4	194.4	359.8	360
50	183.8	240.0	423.8	425
55	202.1	290.3	492.4	495
60	220.5	345.5	566.0	570
65	238.9	405.5	644.4	645
70	257.3	470.3	727.6	730
75	275.6	539.9	815.5	820
80	294.0	614.3	908.3	910

Note: *The above SSD values assume an approach grade less than 3 percent. For downgrades 3 percent* or greater*, see Figure 4.1-C.*

STOPPING SIGHT DISTANCE
(Level Grade)
Figure 4.1-A



STOPPING SIGHT DISTANCE CRITERIA
Figure 4.1-B

4.1.3 Grade Adjustment

The longitudinal gradient of the roadway influences the distance needed for vehicles to brake to a stop. Figure 4.1-C presents the downgrade-adjusted SSD. Where practical, the designer should attempt to meet downgrade-adjusted SSD values.

SSD FOR DOWNGRADES								
Design Speed (mph)	**(3%)**	**(4%)**	**(5%)**	**(6%)**	**(7%)**	**(8%)**	**(9%)**	**(10%)**
15	80	80	81	82	83	84	85	86
20	116	117	119	120	122	124	126	128
25	158	160	162	165	167	170	173	176
30	205	208	211	215	219	223	227	232
35	257	262	266	271	276	282	287	294
40	315	321	327	333	339	347	354	363
45	378	385	392	400	409	418	427	438
50	446	455	464	474	484	495	507	520
55	520	530	541	553	566	579	593	609
60	598	611	624	638	653	669	686	705
65	682	697	712	728	746	765	785	808
70	771	788	806	825	845	868	891	917
75	866	885	906	927	951	976	1003	1033
80	965	987	1011	1035	1062	1091	1121	1155

Notes:

1. *Calculated SSD values are not shown. Values in the table have been rounded up to the next highest 1-foot increment.*

2. *For grades less than 3 percent or upgrades, no adjustment is necessary (i.e., use the level SSD values in Figure 4.1-A).*

3. *For grades intermediate between table values, use a straight-line interpolation to determine the SSD and round up to the next highest 1-foot increment.*

DOWNGRADE-ADJUSTED STOPPING SIGHT DISTANCE
Figure 4.1-C

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4.2 PASSING SIGHT DISTANCE FOR TWO-LANE HIGHWAYS

Where practical on rural new construction/reconstruction two-lane, two-way highway projects, the designer should consider providing passing sight distance over the length of the project. Note that it is generally not cost effective to make significant improvements to the horizontal and vertical alignment solely to increase the available passing sight distance.

4.2.1 Application

On two-lane, two-way highways, vehicles may overtake slower moving vehicles and pass on the lane used by opposing traffic. The necessary passing sight distance for two-lane highways is based on the minimum sight distance for marking no-passing zones as presented in the *Manual on Uniform Traffic Control Devices* (MUTCD). For a discussion on how to determine these distances, the designer should review NCHRP Report 605 *Passing Sight Distance Criteria.*

4.2.2 Design

Figure 4.2-A provides the minimum passing sight distance for design on two-lane, two-way highways. These distances allow the passing vehicle to safely complete the entire passing maneuver.

Design Speed (mph)	Passing Sight Distance (ft)
20	400
25	450
30	500
35	550
40	600
45	700
50	800
55	900
60	1000
65	1100
70	1200
75	1300
80	1400

PASSING SIGHT DISTANCE
(Two-Lane, Two-Way Highways)
Figure 4.2-A

Acceptable passing maneuvers occur when the passing driver can determine that no potentially conflicting vehicles exist before beginning the maneuver. While there may be occasions to consider passing multiple vehicles, it is not practical to assume these conditions in developing minimum design criteria. Instead, the minimum passing sight distances are based on a single vehicle passing a single vehicle.

Passing sight distance is measured from a 3.5-foot height of eye to a 3.5-foot height of object. The 3.5-foot height of object allows the opposing driver to see a sufficient portion of the on-coming vehicle to determine whether to pass.

4.3 DECISION SIGHT DISTANCE

4.3.1 General

At some sites, drivers may be required to make decisions where the highway environment is difficult to perceive or where unexpected maneuvers are required. These are areas of concentrated demand where the roadway elements, traffic volumes and traffic control devices may all compete for the driver's attention. This relatively complex environment may increase the required driver perception/reaction time beyond that provided by the SSD values (i.e., 2.5 seconds) and, in some locations, the desired vehicular maneuver may be a speed/path/direction change rather than a stop. At these locations, the designer should consider providing decision sight distance to provide an additional margin of safety.

Figure 4.3-A provides decision sight distances according to the type of avoidance maneuver. The avoidance maneuvers assumed in the development of Figure 4.3-A are:

- Avoidance Maneuver A: Stop on rural road.
- Avoidance Maneuver B: Stop on urban road.
- Avoidance Maneuver C: Speed/path/direction change on rural road.
- Avoidance Maneuver D: Speed/path/direction change on suburban road.
- Avoidance Maneuver E: Speed/path/direction change on urban road.

4.3.2 Applications

In general, the designer should consider using decision sight distance at any relatively complex location where the driver perception/reaction time may exceed 2.5 seconds. Example locations where decision sight distance may be a factor include:

- freeway exit/entrance gores;
- freeway lane drops;
- freeway left-side entrances or exits;
- intersections near or on a horizontal curve;
- highway/railroad grade crossings;
- approaches to detours and lane closures;
- high-speed, high-volume urban arterials with considerable roadside friction; and/or
- isolated traffic signals on high-speed rural highways.

As with SSD, the driver height of eye is 3.5 feet and the height of object is typically 2.0 feet. Candidate sites (e.g., freeway exit gores) for decision sight distance may also be candidate sites for assuming that the object is the pavement surface (i.e., a 0.0-foot height of object).

Design Speed (mph)	Decision Sight Distance for Avoidance Maneuver (ft)				
	A	B	C	D	E
30	220	490	450	535	620
35	275	590	525	625	720
40	330	690	600	715	825
45	395	800	675	800	930
50	465	910	750	890	1030
55	535	1030	865	980	1135
60	610	1150	990	1125	1280
65	695	1275	1050	1220	1365
70	780	1410	1105	1275	1445
75	875	1545	1180	1365	1545
80	970	1685	1260	1455	1650

Notes:

Avoidance Maneuver A: *Stop on rural road.*
Avoidance Maneuver B: *Stop on urban road.*
Avoidance Maneuver C: *Speed/path/direction change on rural road.*
Avoidance Maneuver D: *Speed/path/direction change on suburban road.*
Avoidance Maneuver E: *Speed/path/direction change on urban road.*

DECISION SIGHT DISTANCE
Figure 4.3-A

4.4 INTERSECTION SIGHT DISTANCE

For an at-grade intersection to operate properly, the designer should provide sufficient sight distance for a driver to perceive potential conflicts and to perform the actions required to negotiate the intersection safely. The additional costs and impacts of removing sight obstructions are often justified.

In general, intersection sight distance (ISD) refers to the corner sight distance available in intersection quadrants that allows a driver at an intersection to observe the actions of vehicles on the crossing leg(s). ISD evaluations involve establishing a sight triangle in each quadrant by determining the legs of the triangle on the two intersecting roadways. The clear sight triangle is based on the type of traffic control at the intersection and on the design speeds of the two roadways. The types of traffic control and maneuvers are as follows:

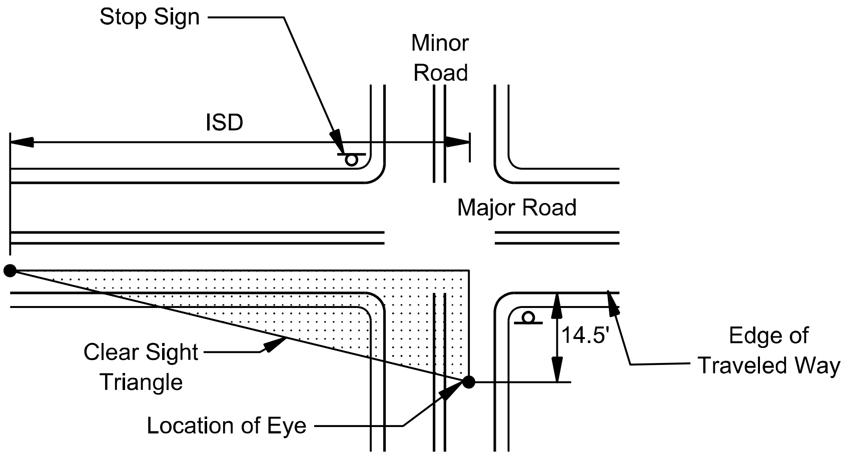
- Case A – Intersections with no control (not used by SCDOT)
- Case B – Intersections with stop control on the minor road:
  - Case B1 – Left-turn from the minor road
  - Case B2 – Right-turn from the minor road
  - Case B3 – Crossing maneuver from the minor road
- Case C – Intersections with yield control on the minor road:
  - Case C1 – Crossing maneuver from the minor road (not used by SCDOT)
  - Case C2 – Left or right-turn from the minor road
- Case D – Intersections with traffic signal control
- Case E – Intersections with all-way stop control
- Case F – Left turns from the major road
- Case G – Roundabout

For guidance on these cases, see the AASHTO *A Policy on Geometric Design of Highways and Streets*, NCHRP Report 383 *Intersection Sight Distance* and NCHRP Report 672 *Roundabouts: An Informational Guide*.

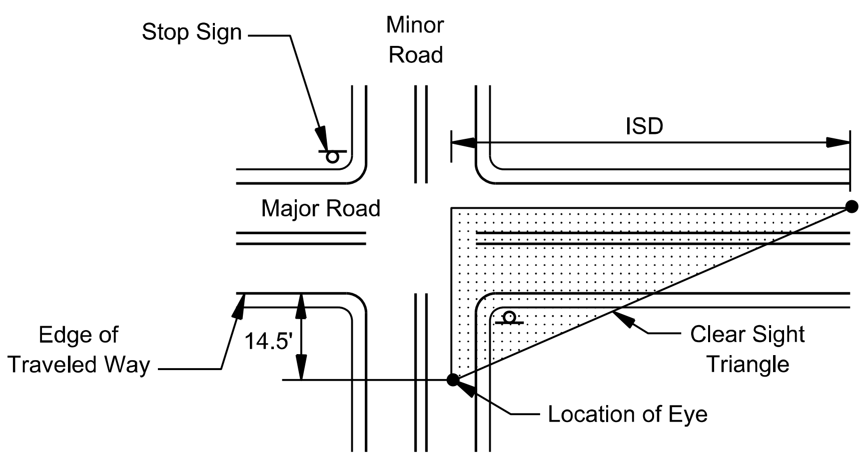
4.4.1 Basic Criteria

The Department uses gap acceptance as the conceptual basis for its ISD criteria at stop-controlled intersections. The intersection sight distance is obtained by providing clear sight triangles both to the right and left as shown in Figure 4.4-A. The lengths of legs of these sight triangles are determined as follows:

1. Minor Road. The length of leg along the minor road is based on two parts. The first is the location of the driver's eye on the minor road. This is typically assumed to be 14.5 feet from the edge of traveled way for the major road and in the center of the lane on the minor road; see Figure 4.4-A. The second part is based on the distance to the center of the vehicle on the major road. For vehicles approaching from the left, this is assumed the center of the closest travel lane from the left. For vehicles approaching from the right, this is assumed to be the center of the closest travel lane for vehicles approaching from the right; see Figure 4.4-A.



a) Clear Sight Triangle For Viewing
Traffic Approaching From The Left



b) Clear Sight Triangle For Viewing
Traffic Approaching From The Right

Note: Pedestrian crossings, design vehicle geometry and other considerations may warrant evaluation of distances greater than 14.5 feet.

CLEAR SIGHT TRIANGLES (STOP-CONTROLLED) INTERSECTIONS
Figure 4.4-A

2. Major Road. The length of the sight triangle leg or ISD along the major road is determined using the following equation:

$$ISD = 1.47\ V_{major}\ t_g \qquad \text{(Equation 4.4-1)}$$

Where:

ISD	=	length of sight line along major road, feet
V_{major}	=	design speed of major road, miles per hour
t_g	=	gap acceptance time for entering the major road, seconds

The gap acceptance time (t_g) varies according to the design vehicle, maneuver type, grade on the minor road approach, number of lanes on the major roadway, type of operation and intersection skew.

3. Height of Eye/Object. The height of eye for passenger cars is assumed 3.5 feet above the surface of the minor road. The height of object (approaching vehicle on the major road) is also assumed to be 3.5 feet. An object height of 3.5 feet assumes that a sufficient portion of the oncoming vehicle must be visible to identify it as an object of concern by the minor road driver. If there are a sufficient number of trucks to warrant their consideration, use an eye height of 7.6 feet. If a truck is the applicable entering vehicle, the object height will still be 3.5 feet for the passenger car on the major road.

Within this clear sight triangle, if practical, the objective is to remove items that obstruct the driver's view. In addition, where a crossroad intersects the major road near a bridge on a crest vertical curve, items such as bridge parapets, piers, abutments, guardrail or the crest vertical curve itself may restrict the clear sight triangle.

4.4.2 Case B – Intersections with Stop Control on the Minor Road

Where traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road should have sufficient sight distance for a safe departure from the stopped position assuming that the approaching vehicle comes into view as the stopped vehicle begins its departure. At a four-leg intersection, the designer should also check the sight distance to cross the intersection.

4.4.2.1 Case B1 – Left-Turn from the Minor Road

To determine the ISD for vehicles turning left onto the major road, the designer should use Equation 4.4-1 and the gap acceptance times (t_g) presented in Figure 4.4-B. This will provide values for sight distance looking right for left-turning vehicles from the minor road as shown in Figure 4.4-A. Figure 4.4-C solves for Equation 4.4-1 and provides the ISD values for left-turning design vehicles onto a two-lane level facility. The designer should also consider the following:

1. Multilane Facilities. For multilane facilities, the gap acceptance times presented in Figure 4.4-B should be adjusted (i.e., add 0.5 second for passenger cars or 0.7 second for trucks for each additional 12-foot lane) to account for the additional distance required by the turning vehicle to cross the additional lanes or median.

2. Medians. The following will apply:

   a. For a multilane facility that does not have a median wide enough to store a design vehicle, divide the median width by 12 feet to determine the lane value (e.g., for a 4-foot median use 0.33), and then use the criteria in Figure 4.4-B to determine the appropriate time factor.

   b. On facilities with a median wide enough to store the design vehicle (e.g., 3 feet clearance at both ends of vehicle as measured from the edge of traveled way), the designer should evaluate the sight distance needed in two separate steps:

      - First, with the vehicle stopped on the side road (the bottom portion in Figure 4.4-D), use the gap acceptance times and distances for a crossing vehicle (Figures 4.4-B and 4.4-C) to determine the applicable ISD. Crossing criteria are discussed in Section 4.4.2.3.

      - Second, with the vehicle stopped in the median (top portion in Figure 4.4-D), assume a two-lane roadway design and use the adjusted gap acceptance times and distances for vehicles turning left (Figures 4.4-B and 4.4-C) to determine the applicable ISD.

3. Approach Grades. If the approach grade on the minor road exceeds 3 percent, add 0.2 seconds to t_g for each percent grade.

4. Design Vehicle. A passenger vehicle is used in most ISD design situations. However, at some intersections (e.g., near truck stops, interchange ramps, schools, grain elevators), the designer should use the single-unit truck or the tractor/ semitrailer design vehicle to determine the ISD. The gap acceptance times (t_g) for passenger cars, single-unit and tractor/semitrailer trucks are provided in Figure 4.4-B. ISD values for level, two-lane roadways are presented in Figure 4.4-C. The eye height for these vehicles is discussed in Section 4.4.1.

Design Vehicle	Gap Acceptance Time (t_g) (sec)
Passenger Car	7.5
Single-Unit Truck	9.5
Tractor/Semitrailer	11.5

Adjustments:

1. *Multilane Highways. For left turns onto two-way multilane highways, add 0.5 second for passenger cars or 0.7 second for trucks for each additional lane from the left, in excess of one, to be crossed by the turning vehicle. Assume that the left-turning driver will enter the left-travel lane on the far side of the major road.*

2. *Minor Road Approach Grades. If the approach upgrade on the minor road exceeds 3 percent, add 0.2 seconds for each percent grade.*

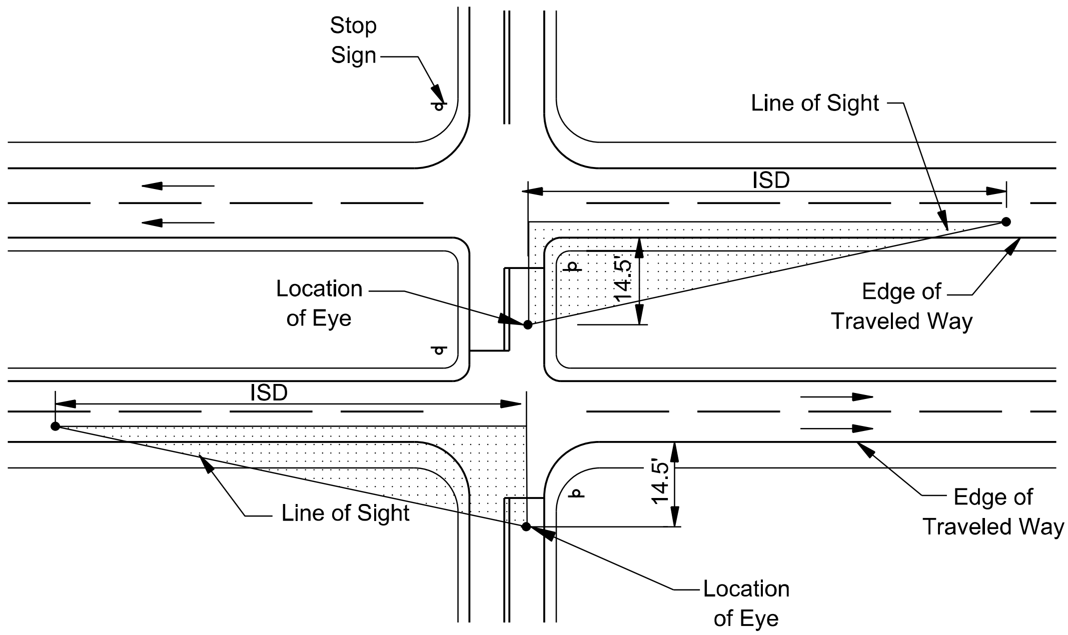
3. *Major Road Approach Grade. Major road grade does not affect calculations.*

GAP ACCEPTANCE TIMES
(Left Turns From Minor Road)
Figure 4.4-B

Design Speed (V_{major}) (mph)	ISD (ft)		
	Passenger Cars	Single-Unit Trucks	Tractor/Semitrailers
15	170	210	255
20	225	280	340
25	280	350	425
30	335	420	510
35	390	490	595
40	445	560	680
45	500	630	765
50	555	700	850
55	610	770	930
60	665	840	1015
65	720	910	1100
70	775	980	1185
75	830	1050	1270
80	885	1120	1355

Note: These ISD values assume a minor road approach grade less than or equal to 3 percent. For grades greater than 3 percent, use the equation with an adjusted t_g.

INTERSECTION SIGHT DISTANCES
(For Left-Turning Vehicle – Sight Distance Towards the Right on a Two-Lane Roadway or Street Only)
Figure 4.4-C



INTERSECTION SIGHT DISTANCE
(Divided Facilities)
Figure 4.4-D

4.4.2.2 Case B2 – Right-Turn from the Minor Road

To determine the ISD for vehicles turning right onto the major road, the designer should use Equation 4.4-1 and the gap acceptance times (t_g) presented in Figure 4.4-E for vehicles on the major road approaching from the left. Note that there are no adjustments required for facilities with medians. The designer should also consider the following:

1. Approach Grades. If the approach grade on the minor road exceeds 3 percent, add 0.1 seconds to t_g for each percent grade.

2. Design Vehicle. A passenger vehicle is used in most ISD design situations. However, at some intersections (e.g., near truck stops, interchange ramps, schools, grain elevators) the designer should use the single-unit truck or the tractor/semitrailer design vehicle to determine the ISD. The gap acceptance times (t_g) for passenger cars, single-unit and tractor/semitrailer trucks are provided in Figure 4.4-E. ISD values for level, two-lane roadways are presented in Figure 4.4-F. The eye height for these vehicles is discussed in Section 4.4.1.

Design Vehicle	Gap Acceptance Time (t_g) (sec)
Passenger Car	6.5
Single-Unit Truck	8.5
Tractor/Semitrailer	10.5

Adjustments:

1. *<u>Multilane Highways</u>. For crossing maneuvers where the design vehicle is crossing a major road with more than two lanes, add 0.5 second for passenger cars or 0.7 second for trucks for each additional lane in excess of two. See the discussion in Section 4.4.5 for additional guidance.*

2. *<u>Approach Grades</u>. If the approach grade on the minor road exceeds 3 percent, add 0.1 second for each percent grade..*

GAP ACCEPTANCE TIMES
(Right Turns from Minor Road and Crossing Maneuvers)
Figure 4.4-E

Design Speed (V_{major}) (mph)	ISD (ft)		
	Passenger Cars	**Single-Unit Trucks**	**Tractor/Semitrailers**
15	145	190	235
20	195	250	310
25	240	315	390
30	290	375	465
35	335	440	545
40	385	500	620
45	430	565	695
50	480	625	775
55	530	690	850
60	575	750	930
65	625	815	1005
70	670	875	1085
75	720	940	1160
80	765	1000	1235

Note: These ISD values apply to the right-turn movement and also to the movement crossing a two-lane facility without a median.

These ISD values assume a minor road approach grade of 3 percent. For grades greater than 3 percent, use the equation with an adjusted t_g.

TWO-LANE INTERSECTION SIGHT DISTANCES
(Right Turns from Minor Road and Crossing Maneuvers)
Figure 4.4-F

4.4.2.3 Case B3 – Crossing Maneuver from the Minor Road

In the majority of cases, the ISD for turning vehicles typically will provide adequate sight distance to allow a vehicle to cross the major road. However, in the following situations, the crossing sight distance may be the more critical movement:

- where left and/or right turns are not permitted from a specific approach and the crossing maneuver is the only legal or expected movement (e.g., indirect left turns);
- where the design vehicle must cross more than six travel lanes or, with medians, the equivalent distance; or
- where a substantial volume of heavy vehicles crosses the highway and there are steep grades on the minor road approach.

Use Equation 4.4-1 and the adjusted gap acceptance times (t_g) in Figure 4.4-E to determine the ISD for crossing maneuvers. Figure 4.4-F presents the applicable ISD values for crossing maneuvers for a level, two-lane highway with no median. Where medians are present, include the median width in the overall length to determine the applicable gap time. Divide this width by 12 feet to determine lane value for the crossing maneuver (e.g., for a 15-foot median use 1.25).

4.4.3 <u>Case D – Intersections with Traffic Signal Control</u>

At intersections with traffic signal control, the first vehicle stopped on one approach should be visible to the driver of the first vehicle stopped on each of the other approaches. Left-turning vehicles should have sufficient sight distance to select gaps in oncoming traffic and complete the left turn (Case F – Section 4.4.5). Check to see if Case B2 ISD (Section 4.4.2.2) is available for right-turning vehicles. If the Case B2 ISD is not available, this may warrant restricting the right-turn-on-red movement.

If the traffic signal is to be placed on two-way flashing operations (i.e., flashing yellow on the major-road approaches and flashing red on the minor-road approaches) under off-peak or nighttime conditions, then the appropriate departure sight triangles for Case B, both to the left and to the right, should be provided for the minor road-approaches.

4.4.4 <u>Case E – Intersections with All-Way Stop Control</u>

For intersections with all-way stop control, provide sufficient sight distance so that the first stopped vehicle on each approach is visible to all other approaches. The ISD criteria for left- or right-turning vehicles as discussed in Section 4.4.2.1 are not applicable in this situation. Often, intersections are converted to all-way stop control to address limited sight distance at the intersection.

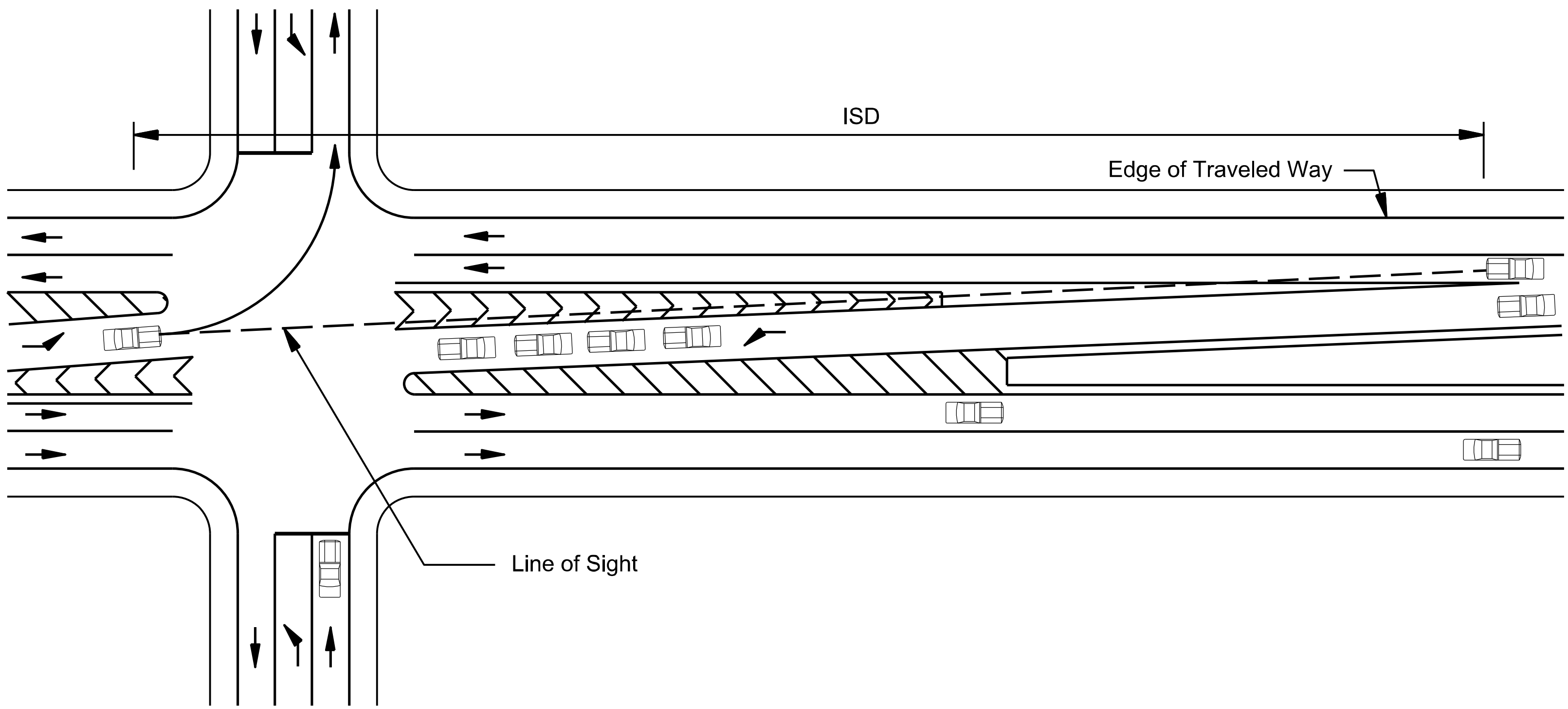
4.4.5 <u>Case F – Left Turns from the Major Road</u>

For all intersections, regardless of the type of traffic control, the designer should consider the sight distance for a stopped vehicle turning left from the major road. The driver will need to see straight ahead for a sufficient distance to turn left and clear the opposing travel lanes before an approaching vehicle reaches the intersection. Sight distance for opposing left turns may be increased by offsetting the left-turn lanes. This situation is illustrated in Figure 4.4-G.

Use Equation 4.4-1 and the gap acceptance times (t_g) from Figure 4.4-H to determine the applicable ISD for the left-turning vehicle. Where the left-turning vehicle must cross more than one opposing lane, add 0.5 second for passenger cars or 0.7 second for trucks for each additional lane in excess of one. Where medians and/or bike lanes are present and the left-turns are not offset, the designer will need to consider the median and/or bike lane width in the same manner as discussed in Section 4.4.2.1. Figure 4.4-I provides the ISD values for typical design vehicles and two common left-turning situations.

4.4.6 Effect of Skew

Where it is impractical to realign an intersection that is greater than 30 degrees from the perpendicular, adjust the gap acceptance times (t_g) presented in the above sections to account for the additional travel time required for a vehicle to make a turn or cross a facility. For oblique-angled intersections, determine the actual path length for a turning or crossing vehicle by dividing the total distance of the lanes and/or median to be crossed by the sine of the intersection angle. If the actual path length exceeds the total width of the lanes to be crossed by 12 feet or more, apply the applicable adjustment factors; see Figure 4.4-J.



Notes:

1. *See Figure 4.4-I for ISD values.*
2. *See Section 4.4.5 for discussion and application.*

INTERSECTION SIGHT DISTANCE FOR A STOPPED VEHICLE TURNING LEFT
(On Major Road)
Figure 4.4-G

Design Vehicle	Gap Acceptance Time (t_g) (sec)
Passenger Car	5.5
Single-Unit Truck	6.5
Tractor/Semitrailer	7.5

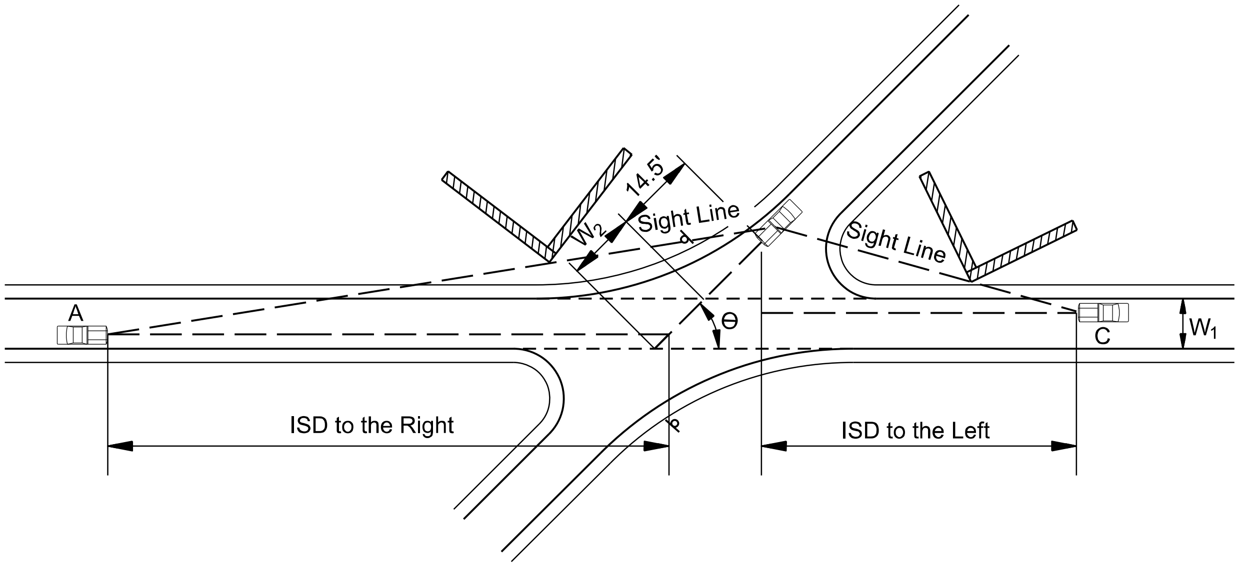
Adjustments: *Where left-turning vehicles cross more than one opposing lane, add 0.5 second for passenger cars or 0.7 second for trucks for each additional lane in excess of one. See Section 4.4.5 for additional guidance on median widths.*

GAP ACCEPTANCE TIMES
(Left Turns from Major Road)
Figure 4.4-H

Design Speed (V_{major}) (mph)	ISD (ft)					
	Passenger Cars		Single-Unit Trucks		Tractor/Semitrailers	
	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes	Crossing 1 lane	Crossing 2 lanes
15	125	135	145	160	170	185
20	165	180	195	215	225	245
25	205	225	240	265	280	305
30	245	265	290	320	335	365
35	285	310	335	375	390	425
40	325	355	385	425	445	485
45	365	400	430	480	500	545
50	405	445	480	530	555	605
55	445	490	530	585	610	665
60	490	530	575	640	665	725
65	530	575	625	690	720	785
70	570	620	670	745	775	845
75	610	665	720	795	830	905
80	650	710	765	850	885	965

Note: *Assumes no median on major road.*

INTERSECTION SIGHT DISTANCES
(Left Turns from Major Road)
Figure 4.4-I



$$W_2 = \frac{W_1}{\sin\theta}$$

Where: W_1 = Major road traveled way width (feet)
W_2 = Adjusted width for skew (feet)
θ = Intersection angle (degrees)

SIGHT DISTANCE AT SKEWED INTERSECTIONS
Figure 4.4-J

4.4.7 Examples of ISD Applications

The following examples illustrate the application of the ISD criteria:

* * * * * * * * * *

Example 4.4-1

Given: Minor road intersects a four-lane highway with a TWLTL.
Minor road is stop controlled and intersects major road at 90 degrees.
Design speed of the major highway is 45 miles per hour.
All travel lane widths are 12 feet.
The TWLTL width is 15 feet.
Grade on minor road is 1 percent.
Trucks are not a concern.

Problem: Determine the ISD to the left and right from the minor road.

Solution: The following steps will apply:

1. For the passenger car turning right, the ISD to the left can be determined directly from Figure 4.4-F. For the 45 miles per hour design speed, the ISD to the left is 430 feet.

2. For the passenger car turning left, the ISD to the right must reflect the additional time required to cross the additional lanes and TWLTL; see Section 4.4.2.1. The following will apply:

   a. First, determine the extra width required by the one additional travel lane and the TWLTL and divide this number by 12 feet:

   $$\frac{(12+15)}{12} = 2.25 \text{ lanes}$$

   b. Next, multiply the number of lanes by 0.5 second to determine the additional time required:

   (2.25 lanes)(0.5 sec/lane) = 1.125 seconds

   c. Add the additional time to the basic gap time of 7.5 seconds and insert this value into Equation 4.4-1:

   ISD = (1.47)(45)(7.5 + 1.125) = 570.5 feet

   Provide an ISD of 575 feet to the right for the left-turning vehicle.

3. Check the crossing vehicle, as discussed in Section 4.4.2.3. The following will apply:

   a. First determine the extra width required by the two additional travel lanes and the TWLTL and divide this number by 12 feet:

   $$\frac{(12+12+15)}{12} = 3.25 \text{ lanes}$$

   b. Next, multiply the number of lanes by 0.5 second to determine the additional time required:

   (3.25 lanes)(0.5 sec/lane) = 1.625 seconds

   c. Add the additional time to the basic gap time of 6.5 seconds and insert this value into Equation 4.4-1:

   ISD = (1.47)(45)(6.5 + 1.625) = 540 feet

   The 540 feet for the crossing maneuver is less than the 575 feet required for the left-turning vehicle and, therefore, is not the critical maneuver.

4. For the passenger car turning left, the ISD to the left can be determined directly from Figure 4.4-F. For the 45 miles per hour design speed, the ISD to the left is 430 feet.

Example 4.4-2

Given: Minor road intersects a two-lane highway with a TWLTL.
Minor road is stop controlled and intersects major road at 90 degrees.

There are 4-foot bike lanes on both sides of the major road.
Design speed of the major highway is 30 miles per hour.
All travel lane widths are 12 feet.
The TWLTL width is 15 feet.
Grade on minor road is 1 percent.
Trucks are not a concern.

Problem: Determine the ISD to the left and right from the minor road.

Solution: The following steps will apply:

1. For the passenger car turning right, the ISD to the left can be determined directly from Figure 4.4-F. For the 30 miles per hour design speed, the ISD to the left is 290 feet.

2. For the passenger car turning left, the ISD to the right must reflect the additional time required to cross the additional bike lane and TWLTL; see Section 4.4.2.1. The following will apply:

   a. First, determine the extra width required by the additional bike lane and the TWLTL and divide this number by 12 feet:

   $$\frac{(4+15)}{12} = 1.58 \text{ lanes}$$

   b. Next, multiply the number of lanes by 0.5 second to determine the additional time required:

   (1.58 lanes)(0.5 sec/lane) = 0.79 seconds

   c. Add the additional time to the basic gap time of 7.5 seconds and insert this value into Equation 4.4-1:

   ISD = (1.47)(30)(7.5 + 0.79) = 365.6 feet

   Provide an ISD of 370 feet to the right for the left-turning vehicle.

3. Check the crossing vehicle, as discussed in Section 4.4.2.3. The following will apply:

   a. First determine the extra width required by the two additional bike lanes and the TWLTL and divide this number by 12 feet:

   $$\frac{(4+4+15)}{12} = 1.92 \text{ lanes}$$

   b. Next, multiply the number of lanes by 0.5 second to determine the additional time required:

   (1.92 lanes)(0.5 sec/lane) = 0.96 seconds

   c. Add the additional time to the basic gap time of 6.5 seconds and insert this value into Equation 4.4-1:

$$ISD = (1.47)(30)(6.5 + 0.96) = 329.0 \text{ feet}$$

Use 330 feet. The 330 feet for the crossing maneuver is less than the 370 feet required for the left-turning vehicle and, therefore, is not the critical maneuver.

Example 4.4-3

Given: Minor road intersects a four-lane divided highway.
Minor road is stop controlled and intersects major road at 90 degrees.
Design speed of the major highway is 60 miles per hour.
All travel lane widths are 12 feet.
The median width is 50 feet.
Grade on minor road is 4 percent.
Grade in median is 1 percent.
The design vehicle is a 64-passenger school bus that is 35.8 feet long.

Problem: Determine the ISD to the left and right from the minor road.

Solution: The following steps apply:

For a school bus, assume an SU design vehicle.

1. For the school bus turning right, the ISD to the left can be determined using Equation 4.4-1. Because the grade exceeds 3%, add 0.1 seconds to the 8.5 seconds for the school bus time gap. Using Equation 4.4-1 directly:

   $$ISD = (1.47)(60)(8.5 + 0.01) = 858.5 \text{ feet}$$

   Provide an ISD of 860 feet to the left for the right-turning vehicle.

2. Determine if the crossing maneuver is critical; see Section 4.4.2.3. Also, because the approach is greater than 3 percent, the gap time needs to be increased by 0.1 second for the additional grade (4% − 3%). Using Equation 4.4-1 directly and Figure 4.4-F:

   $$ISD = (1.47)(60)(8.5+.01) = 858.5 \text{ feet; use 860 feet}$$

   The crossing maneuver ISD is the same distance as the right-turning maneuver.

3. For the school bus turning left, it can be assumed the school bus can safely stop in the median (i.e., 50 feet minus 35.8 feet). The ISD to the right can be determined directly from Figure 4.4-C. For the 60 miles per hour design speed, the ISD to the right for the left turn is 840 feet. The crossing maneuver will not be critical.

Example 4.4-4

Given: Minor road intersects a four-lane divided highway.
Minor road is stop controlled and intersects major road at 90 degrees.
Design speed of the major highway is 50 miles per hour.
All travel lane widths are 12 feet.

Existing median width is 24 feet.
Turn occurs from the 12 foot left-turn lane.
Trucks are not a concern.

Problem: Determine the ISD for a vehicle turning left from the major road.

Solution:

The median is too narrow to store the turning vehicle. Therefore, the turning movement must be made in one motion. For the passenger car turning left, the ISD must reflect the additional time required to cross the median and additional lanes; see Section 4.4.5. The following will apply:

1. First, determine the extra width required by the one additional travel lane and the median width minus the left-turn lane and divide this number by 12 feet:

$$\frac{(12+24-12)}{12} = 2\,\text{lanes}$$

2. Next, multiply the number of lanes by 0.5 second to determine the additional time required:

   (2 lanes)(0.5 sec/lane) = 1.0 seconds

3. Add the additional time to the basic gap time of 5.5 seconds and insert this value into Equation 4.4-1:

   ISD = (1.47)(50)(5.5 + 1.0) = 477.8 feet

   Provide an ISD of 480 feet for the left-turning vehicle.

* * * * * * * * * *

4.5 SIGHT DISTANCES AT ROUNDABOUTS

Sight distance must also be verified for any roundabout design to ensure that sufficient distance is available for drivers to perceive and react to the presence of conflicting vehicles, pedestrians and bicyclists.

4.5.1 Stopping Sight Distance

The stopping sight distance provided in Figure 4.1-A should be provided at every point within a roundabout and on each entering and exiting approach. When applying the SSD values, the height of eye is assumed to be 3.5 feet and the height of object 2 feet. The designer should check the stopping sight distances at the following critical locations (see Figure 4.5-A):

- approach sight distance (Figure 4.5-A(a)),
- circulatory roadway sight distance (Figure 4.5-A(b)), and
- sight distance to crosswalk on exit (Figure 4.5-A(c)).

4.5.2 Intersection Sight Distance

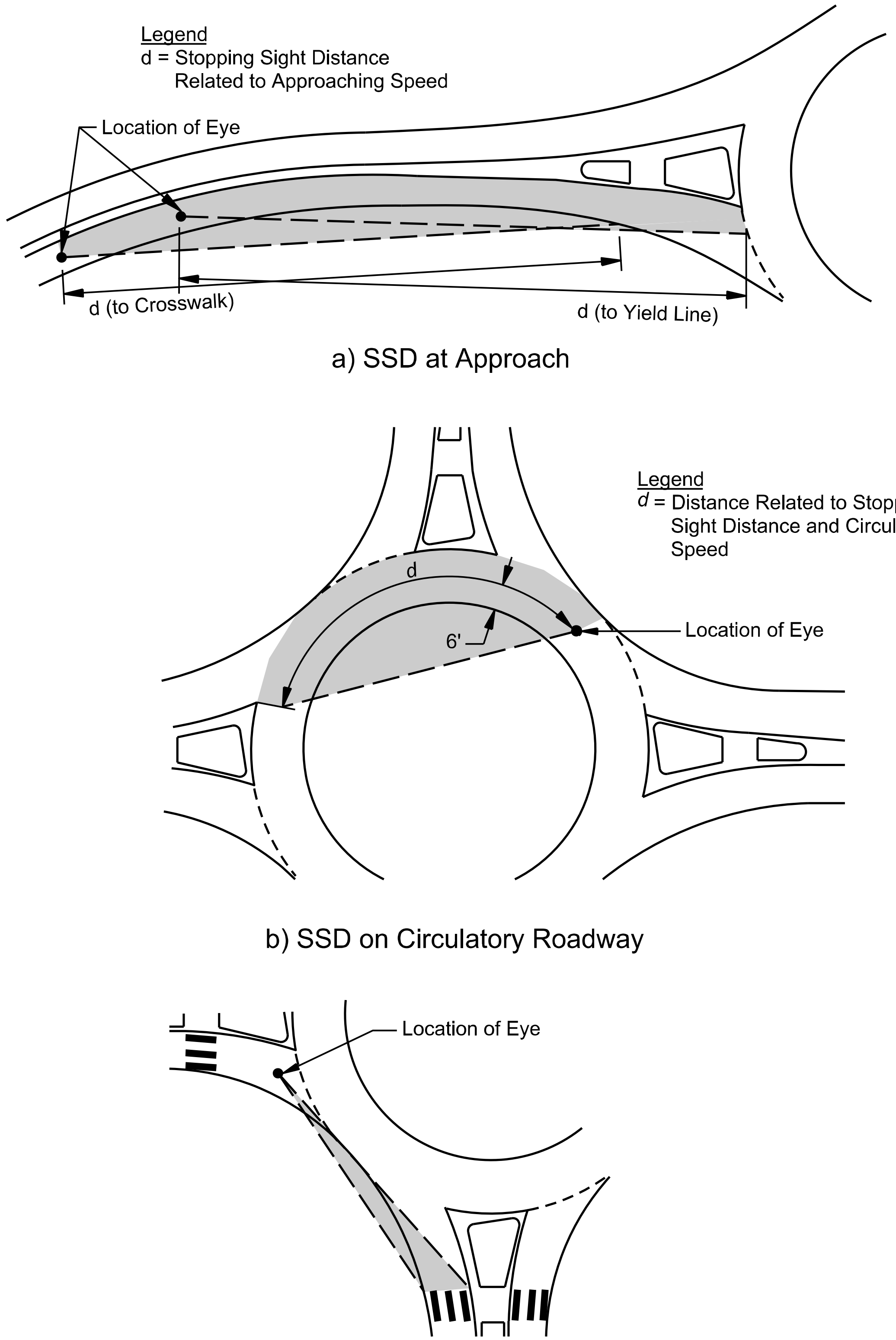
Intersection sight distance is determined by the use of the sight triangle. This triangle is bounded by a length of roadway defining a limit away from the intersection on each of the two conflicting approaches and by a line connecting those two limits. For roundabouts, these legs follow the curvature of the roadway and, therefore, the distances are measured along the vehicular path; see Figure 4.5-B. The height of eye for passenger cars is assumed 3.5 feet above the pavement surface. The height of object is also assumed to be 3.5 feet. There are two conflicting approaches that must be checked independently.

1. Approach Leg. The length of the approach leg of the sight triangle should be limited to 50 feet. Research has shown that greater distances results in a higher frequency of crashes.

2. Conflicting Legs. A vehicle approaching an entry to a roundabout faces conflicting vehicles within the circulatory roadway and on the immediate upstream entry. The critical headway for entering the major road (t_c) is assumed to be 5.0 seconds for passenger cars. T_c is substituted for gap acceptance time in Equation 4.4-1 to calculated ISD. To determine the design speed for each conflicting stream, the designer should use the following:

    a. Entering Stream. The entering stream is composed of vehicles from the immediate upstream entry. The speed of this movement can be approximated by taking the average of the entering speed and the circulating speed.

    b. Circulating Stream. The circulating stream is composed of vehicles that enter the roundabout prior to the immediate upstream entry. This speed can be approximated by the speed of left-turning vehicle.

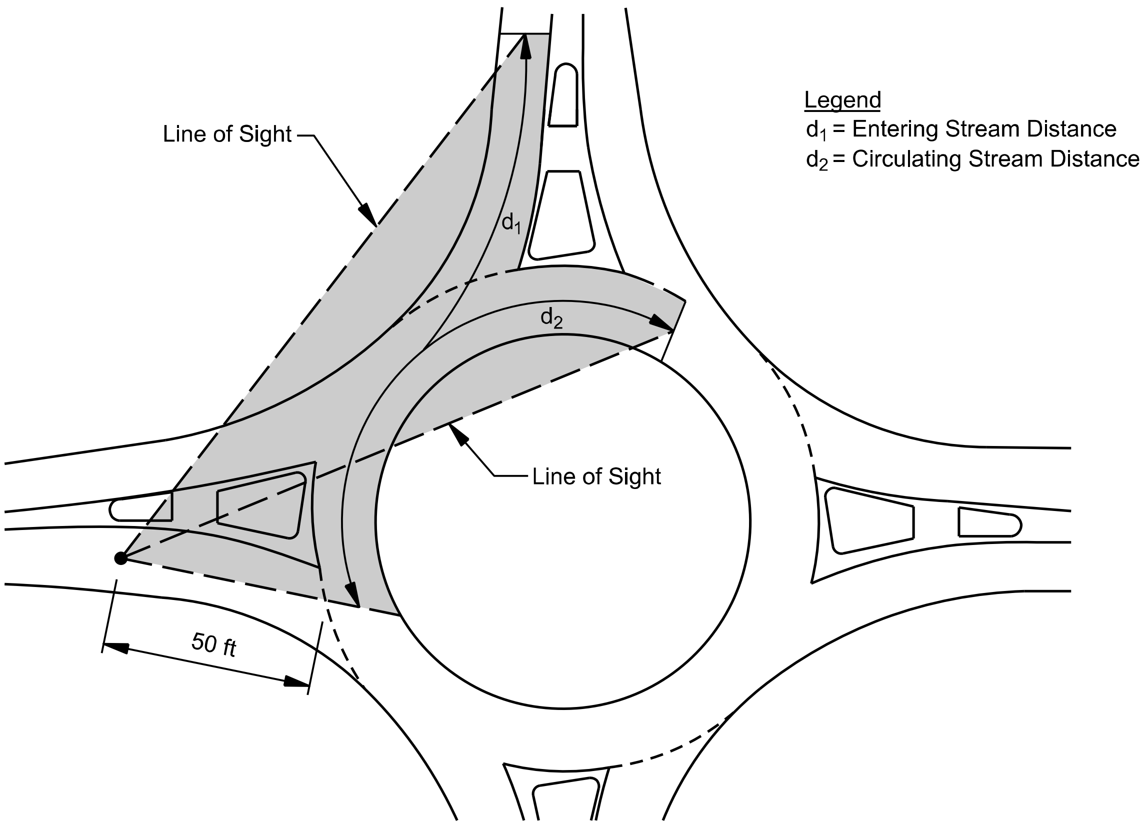


a) SSD at Approach

b) SSD on Circulatory Roadway

c) SSD at Exit

STOPPING SIGHT DISTANCE AT ROUNDABOUTS
Figure 4.5-A



INTERSECTION SIGHT DISTANCE AT ROUNDABOUTS
Figure 4.5-B

4.5.3 Example of ISD at Roundabouts

The following example illustrates how to determine ISD at a roundabout.

* * * * * * * * * *

Example 4.5-1

Given: Two urban collectors intersect with a single-lane roundabout.
Each collector has a design speed of 35 miles per hour.
The entry to the roundabout is designed with a 25 miles per hour design speed.
The circulatory design speed within the roundabout is 20 miles per hour.

Problem: Determine the ISD for the two critical movements within the roundabout.

Solution: The following steps will apply:

1. As discussed in Section 4.5.2, first determine the ISD from the entry approach to the first entry approach to the left (d_1 in Figure 4.5-B). The following will apply:

   a. First, determine the average of the entry speed and circulatory speed:

$$\frac{(25+20)}{2}=22.5\text{ mph}$$

b. Use this average speed and the gap time of 5 seconds in Equation 4.1-1:

ISD = (1.47)(22.5)(5.0) = 165.4 feet

Provide an ISD of 170 feet to the first entry point on the left.

2. Next, check the ISD within the circulatory road (d_2 in Figure 4.5-B) using Equation 4.1-1 and the gap time of 5.0 seconds:

ISD = (1.47)(20)(5.0) = 147.0 feet

Ensure that 150 feet available to a vehicle traveling within the roundabout.

4.6 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. NCHRP Report 400 *Determination of Stopping Sight Distances*, Transportation Research Board, 1997.

3. *Manual on Uniform Traffic Control Devices,* FHWA, ATSSA, AASHTO, ITE, 2009*.*

4. NCRP Report 605 *Passing Sight Distance Criteria*, Transportation Research Board, 2007.

5. NCHRP Report 383 *Intersection Sight Distance*, Transportation Research Board, 1996.

6. NCHRP Report 672 *Roundabouts: An Informational Guide (Second Edition),* Transportation Research Board, 2010.

SPACER PAGE

Chapter 5

HORIZONTAL ALIGNMENT

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

SPACER PAGE

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Section **Page**

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(Continued)

Chapter 5
HORIZONTAL ALIGNMENT

This chapter presents SCDOT criteria for the design of horizontal alignment elements. This includes horizontal curvature, superelevation and sight distance around horizontal curves.

5.1 DEFINITIONS

The following presents definitions for the basic elements of horizontal alignment:

1. Broken-Back Curves. Closely spaced horizontal curves with deflection angles in the same direction with an intervening, short tangent section (less than 1500 feet).

2. Compound Curves. A series of two or more simple curves with deflections in the same direction immediately adjacent to each other without a tangent section.

3. Design Superelevation (e_d). The amount of cross slope or bank provided on a horizontal curve to counterbalance, in combination with the side friction, the centrifugal force of a vehicle traversing the curve.

4. Low-Speed Urban Streets. All streets within urbanized or small urban areas with a design speed of 45 miles per hour or less.

5. Maximum Relative Gradient (Δ). For superelevation transition sections on two-lane facilities, the relative gradient between the centerline profile grade and edge of traveled way grade.

6. Maximum Side Friction (f_{max}). Limiting values selected by AASHTO for use in the design of horizontal curves. The designated f_{max} values represent a threshold of driver discomfort and not the point of impending skid.

7. Maximum Superelevation (e_{max}). An overall superelevation control used on a widespread basis. Its selection depends on several factors including type of area (rural or urban), design speed and climate.

8. Normal Crown (NC). The typical cross slope on a tangent section of roadway (i.e., no superelevation).

9. Open-Roadway Conditions. Rural facilities for all design speeds and urban facilities with a design speed greater than 45 miles per hour.

10. Point of Revolution. The point about which the pavement is revolved to superelevate the roadway.

11. Remove Adverse Crown (RC). A superelevated roadway section that is sloped across the entire traveled way in the same direction and at a rate equal to the cross slope on the tangent section (typically 2.00 percent).

12. Reverse Curves. Two simple curves with deflections in opposite directions that are joined by a relatively short tangent distance or that have no intervening tangent (i.e., the PT and PC are at the same point).

13. Side Friction (f). The interaction between the tire and the pavement surface to counterbalance, in combination with the superelevation, the centrifugal force or lateral acceleration of a vehicle traversing a horizontal curve.

14. Simple Curves. Continuous arcs of constant radius that achieve the necessary highway deflection without an entering or exiting transition.

15. Superelevation Transition Length. The distance required to transition the roadway from a normal crown section to the design superelevation rate. Superelevation transition length is the sum of the tangent runout and superelevation runoff distances:

    a. Tangent Runout (L_t). The length of roadway needed to accomplish a change in outside-lane cross slope from the normal cross slope rate to zero (flat), or vice versa.

    b. Superelevation Runoff (L_r). The length of roadway needed to accomplish a change in outside-lane cross slope from zero (flat) to the design superelevation (e_d), or vice versa.

5.2 HORIZONTAL CURVES

Horizontal curves are, in effect, transitions between two tangents. These deflectional changes are necessary in virtually all highway alignments to avoid impacts on a variety of field conditions (e.g., right of way, natural features, man-made features). Figure 5.2-A illustrates a simple horizontal curve.

5.2.1 General Theory

This section briefly summarizes the theoretical basis for the design of horizontal curves. For more information, the designer should review the latest edition of AASHTO *A Policy on Geometric Design of Highways and Streets*.

5.2.1.1 Basic Curve Equation

The point-mass formula is used to define vehicular operation around a curve. Where the curve is expressed using its radius, the basic equation for a simple curve is:

$$R = \frac{V_d^2}{15(e_d + f)} \qquad \text{(Equation 5.2-1)}$$

Where:

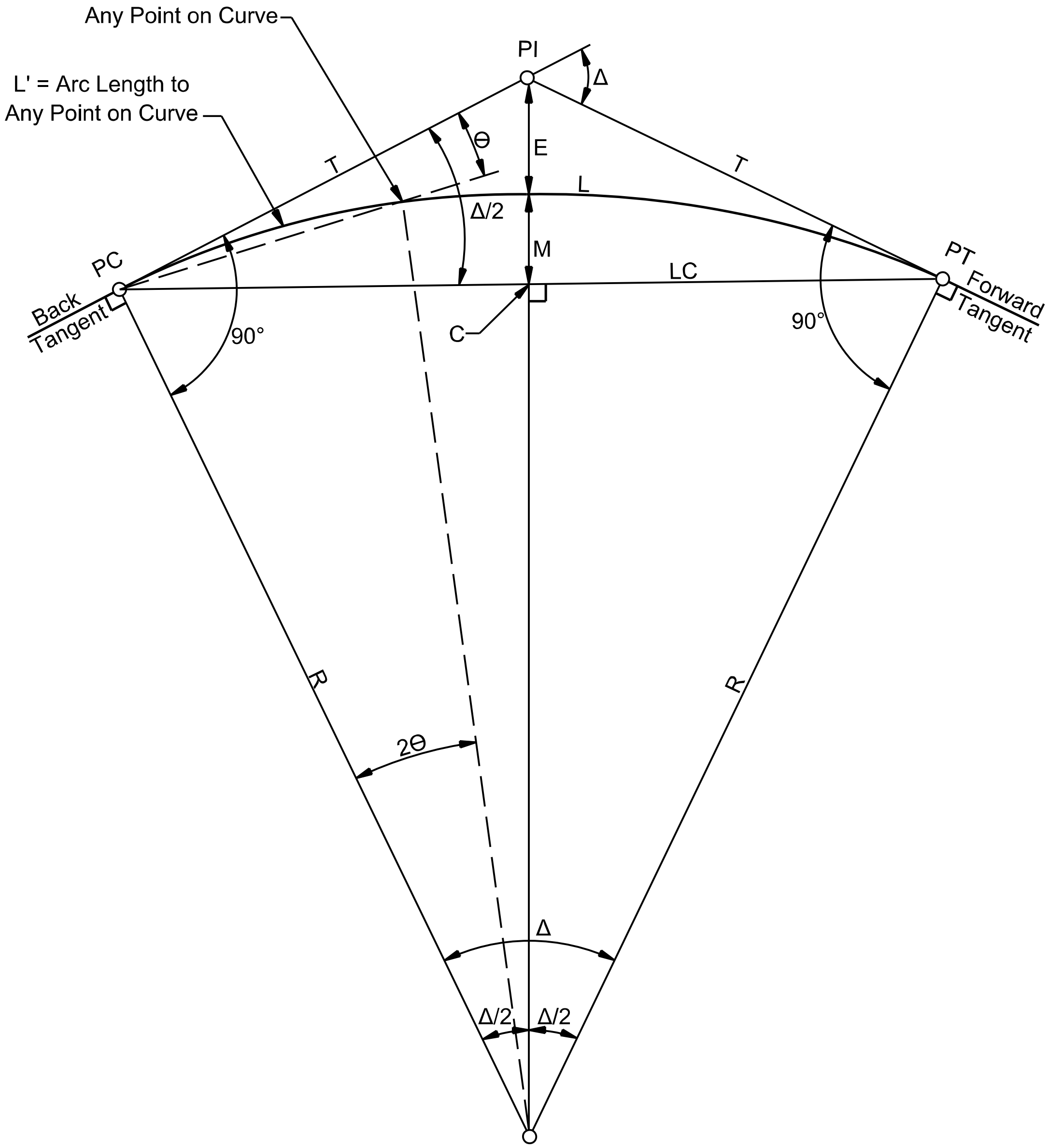
R	=	radius of curve, feet
e_d	=	design superelevation rate, decimal
f	=	side-friction factor, decimal
V_d	=	design speed, miles per hour

To convert to the degree-of-curve definition for a horizontal curve, use the following equation:

$$D = \frac{100 \times 360}{2\pi R} = \frac{5729.58}{R} \qquad \text{(Equation 5.2-2)}$$

Where:

D	=	degree of curve, degrees
R	=	radius of curve, feet



Notes:

PI = Point of Intersection of Tangents
PC = Point of Curvature (Beginning of Curve)
PT = Point of Tangency (End of Curve)
R = Radius of Curve, feet
C = Mid-point of Long Chord
Δ = Deflection Angle Between Tangents or Central Angle, degrees
T = Tangent, Distance, feet
LC = Length of Long Chord, feet
L = Length of Curve, feet
E = External Distance, feet
M = Middle Ordinate

$$E = T \tan\left(\frac{\Delta}{4}\right)$$

$$T = R \tan\left(\frac{\Delta}{2}\right)$$ *where Δ is expressed as a decimal*

$$LC = 2T\left(\cos\left(\frac{\Delta}{2}\right)\right)$$

$$M = R\left(1 - \cos\left(\frac{\Delta}{2}\right)\right)$$

$$L = \frac{\Delta R}{57.2958}$$

SIMPLE HORIZONTAL CURVE ELEMENTS
Figure 5.2-A

5.2.1.2 Theoretical Approaches

Establishing horizontal curvature criteria requires a determination of the theoretical basis for the various factors in the basic curvature equations. These include the side-friction factor (f) and the distribution method between side friction and superelevation. The theoretical basis will be one of the following:

1. Open-Roadway Conditions. The theoretical basis for horizontal curvature assuming open-roadway conditions includes the use of AASHTO Method 5 to distribute side friction and superelevation; see Section 5.3.2. Open-roadway conditions apply to all rural facilities and all urban facilities where the design speed (V_d) > 45 miles per hour.

2. Low-Speed Urban Streets. The theoretical basis for horizontal curvature assuming low-speed urban street conditions includes the use of AASHTO Method 2 to distribute side friction and superelevation; see Section 5.3.3. Low-speed urban streets are defined as streets within an urban or urbanized area where the design speed (V_d) ≤ 45 miles per hour.

5.2.1.3 Superelevation

Superelevation allows a driver to negotiate a curve at a higher speed than would otherwise be comfortable. Superelevation and side friction work together to offset the outward pull of the vehicle as it traverses the horizontal curve. In highway design, it is necessary to establish limiting values of superelevation (e_{max}) based on the operational characteristics of the facility. See Section 5.3.1.

5.2.1.4 Side Friction

AASHTO has established limiting side-friction factors (f) for various design speeds. It is important to understand that the f values used in design represent a threshold of driver discomfort and not the point of impending skid. See Figures 5.2-B through 5.2-D for maximum design f values.

5.2.2 Types of Horizontal Curves

5.2.2.1 General

This section discusses the types of horizontal curves that may be used to achieve the necessary roadway deflection. For each type, the discussion briefly describes the curve and presents the SCDOT usage of the curve type.

Design Speed, (V_d) (miles per hour)	**f_{max}**	**Minimum Radii, R_{min}* (feet)**
25	0.23	134
30	0.20	214
35	0.18	314
40	0.16	444
45	0.15	587
50	0.14	758
55	0.13	960
60	0.12	1200
65	0.11	1480
70	0.10	1810
75	0.09	2210

$$*R_{min} = \frac{V_d^{\,2}}{15(e_{max} + f_{max})}$$

MINIMUM RADII (e_{max} = 8.0 Percent)
Figure 5.2-B

Design Speed, (V_d) (miles per hour)	**f_{max}**	**Minimum Radii, R_{min}* (feet)**
20	0.27	81
25	0.23	144
30	0.20	231
35	0.18	340
40	0.16	485
45	0.15	643
50	0.14	833

$$*R_{min} = \frac{V_d^{\,2}}{15(e_{max} + f_{max})}$$

MINIMUM RADII (e_{max} = 6.0 Percent)
Figure 5.2-C

Design Speed, (V_d) (miles per hour)	**f_{max}**	**Minimum Radii, R_{min}* (feet)**
20	0.27	86
25	0.23	154
30	0.20	250
35	0.18	371
40	0.16	533
45	0.15	711

$$*R_{min} = \frac{V_d^{\,2}}{15(e_{max} + f_{max})}$$

MINIMUM RADII (e_{max} = 4.0 Percent)
Figure 5.2-D

5.2.2.2 Simple Curves

Simple curves are continuous arcs of constant radius that achieve the necessary roadway deflection without an entering or exiting transition. The radius (R) defines the circular arc that a simple curve will transcribe. All angles and distances for simple curves are computed in a horizontal plane; see Figure 5.2-A.

Because of their simplicity and ease of design, survey and construction, SCDOT typically uses the simple curve design on its highways.

5.2.2.3 Compound Curves

Compound curves are a series of two or more simple curves of different radii with deflections in the same direction. Exercise caution when considering the use of compound curves and avoid their use where curves are sharp. Compound curves with large differences in radius introduce the same concerns that arise at tangent approaches to circular curves. SCDOT only uses compound curves on a roadway mainline to maintain a desired alignment or to meet field conditions (e.g., to avoid obstructions that cannot be relocated, restricted right of way) where a simple curve is not applicable. When a compound curve is used on a highway mainline, the radius of the flatter circular arc (R_1) should not be more than 50 percent greater than the radius of the sharper circular arc (R_2); i.e., $R_1 \leq 1.5\ R_2$. These design guidelines for compound curves are developed on the premise that travel is in the direction of sharper curvature. For the acceleration condition, the 2:1 ratio is not as critical and may be exceeded.

Chapter 9 “Intersections” discusses the use of compound curves for intersections (e.g., curb radii, turning roadways). Chapter 10 “Interchanges” discusses the use of compound curves on interchange ramps.

5.2.2.4 Reverse Curves

Reverse curves are two simple curves with deflections in opposite directions that are joined by a relatively short tangent distance. Avoid abrupt reversals in alignment. Such changes in alignment make it difficult for drivers to keep within their own lane. It is also difficult to superelevate both curves adequately, and erratic operations may result. See Section 5.3.7 for superelevation development of reverse curves. The distance between reverse curves should be the sum of the superelevation runoff lengths and the tangent runout lengths. If sufficient distance (e.g., more than 300 feet) is not available to permit the tangent runout lengths, there may be a significant portion of the roadway where the centerline and the edges of roadway are at the same elevation and poor transverse drainage is likely. In this case, increase the superelevation runoff lengths until they adjoin (i.e., there is only one instantaneous level section). In rural areas, desirably provide at least 500 feet between the PT and PC of the two curves for appearance.

5.2.2.5 Broken-Back Curves

Broken-back curves are closely spaced horizontal curves with deflection angles in the same direction with an intervening, short tangent section (less than 1500 feet) from PT to PC.

Desirably, limit the use of broken-back curves on the highway mainline for the following reasons:

- Except on circumferential highways, most drivers do not expect successive curves to be in the same direction, which may confuse a driver.
- Broken-back curves may cause problems with superelevation development (i.e., they may require continuous superelevation).
- Broken-back alignments are not pleasing in appearance. Even if the tangent between the curves is of considerable length, the alignment may be unpleasant in appearance if both curves are clearly visible.

5.2.3 Minimum Radii

The minimum radius is calculated using Equation 5.2-1 and the applicable values of e_{max} and f_{max}. See Figures 5.2-B, 5.2-C and 5.2-D. In most cases, the designer should limit the use of minimum radii because this results in the use of maximum superelevation rates.

5.2.4 Maximum Deflection Without Curve

As a guide, the designer may retain deflection angles of approximately 1 degree or less (urban) and 0 degree 30 minutes or less (rural) on the roadway mainline. For these angles, the absence of a horizontal curve should not affect aesthetics. The larger deflection angles for lane shifts that are based on the formulas L=WS or $L=WS^2/60$ are acceptable where the through lanes are shifted to add turn lanes at intersections. See Section 9.5.3 for additional guidance.

5.2.5 Minimum Length of Curve

For small deflection angles, horizontal curves should be sufficiently long to avoid the appearance of a kink. For aesthetics, a minimum 500-foot length of curve for a 5-degree central angle will eliminate the sense of abruptness. Where the central angle is less than 5 degrees, see Figure 5.2-E for the minimum length of curve. The designer should also consider the following:

1. Urban Streets. The minimum length of curves on low-speed urban streets will be determined on a case-by-case basis.
2. All Major Highways. The minimum length of curve should be $15V_d$, where V_d equals the design speed in mph.
3. Freeways. For aesthetics, it is desirable that the minimum length of curve be $30V_d$.

Deflection Angle	Minimum Length (Feet)
5°00′	500
4°30′	550
4°00′	600
3°30′	650
3°00′	700
2°30′	750
2°00′	800

MINIMUM LENGTHS OF CURVE
Figure 5.2-E

5.2.6 Traveled Way Widening

Traveled way widening may be considered on the inside edge of horizontal curves on two-lane highways for the following reasons:

1. Vehicles (especially trucks) occupy a greater effective width because rear wheels track inside of front wheels when rounding a curve.

2. Known problem areas (e.g., where the inside shoulder has broken up) may warrant widening.

To determine if widening is required, the designer should only consider the largest expected vehicle that will routinely use the road or street. Where used, apply the widening to the inside edge of the pavement. The transition distance for traveled way widening should equal the superelevation transition length and be applied with the superelevation transition. See the *SCDOT Standard Drawings* for further design guidance.

5.2.7 Design Controls

As discussed elsewhere in Chapter 5, the design of horizontal alignment involves, to a large extent, complying with specific limiting criteria. These include minimum radii, superelevation rates and sight distance around curves. In addition, the designer should adhere to certain design principles and controls that will determine the overall safety of the facility and will enhance the aesthetic appearance of the highway. These design principles include:

1. Consistency. Alignment should be consistent. Avoid sharp curves at the ends of long tangents and sudden changes from gentle to sharply curving alignment.

2. Directional. Alignment should be as directional as practical and consistent with physical and economic constraints. On divided highways, a flowing line that conforms generally to the natural contours is preferable to one with long tangents that slash through the terrain. Directional alignment will be achieved by using the smallest practical central angles.

3. Use of Minimum Radii. Do not use minimum radii, if practical, especially in level terrain.

4. High Fills. Avoid sharp curves on long, high fills. Under these conditions, it is difficult for drivers to perceive the extent of horizontal curvature.

5. Alignment Reversals. Avoid abrupt reversals in alignment (reverse curves). Provide a sufficient tangent distance between the curves to ensure proper superelevation transitions for both curves and to allow time for the motorist to perceive the next decision point.

6. Broken-Back Curvature. Avoid broken-back curves. This arrangement is not aesthetically pleasing, violates driver expectancy and may create undesirable superelevation development requirements.

7. Compound Curves. Limit the use of compound curves on highway mainline.

8. Coordination with Natural/Man-Made Features. The designer should coordinate the horizontal alignment with the existing alignment at the ends of new projects, natural topography, available right of way, utilities, roadside development and natural/man-made drainage patterns.

9. Environmental Impacts. Horizontal alignment should be properly coordinated to avoid or minimize environmental impacts (e.g., encroachment onto wetlands).

10. Intersection and Travel Lane Tapers. Horizontal alignment through intersections presents special considerations (e.g., intersection sight distance, superelevation development, crossover crowns). See Chapter 9 “Intersection” for the design of intersections and tapers.

11. Coordination with Vertical Alignment. Section 6.2.2 discusses general design principles for the coordination between horizontal and vertical alignments.

12. Bridges. Horizontal alignment must be coordinated with the location of bridges. Evaluate the need for curvature and superelevation development for each bridge location. Avoid superelevation transitions on bridges, if practical, to facilitate bridge design and construction. Also, consider crossing angles between the mainline and other features. It is preferable for the mainline to intersect other features at right angles. Skewed crossing can impact the structural design of bridges and skewed bridges typically are more expensive to design and to construct. In addition, the structural response of a skewed bridge to seismic and thermal loads can be significantly altered by the skew angle of the bridge substructure. When a skewed crossing cannot be avoided, strive to minimize the angle of the skew.

5.3 SUPERELEVATION

The *SCDOT Standard Drawings* includes a graphical presentation of typical Department practices for superelevation development (e.g., point of revolution, distribution of superelevation between tangent and curve). Section 5.3 provides an elaboration on SCDOT's superelevation practices.

5.3.1 General

5.3.1.1 Distribution of Superelevation and Side Friction

As discussed in Section 5.2.3, the minimum radius is based on e_{max} and f_{max} that apply to the facility. For curvature flatter than the minimum, a methodology must be applied to distribute superelevation and side friction for a given radius and design speed. The following describes these distribution methods:

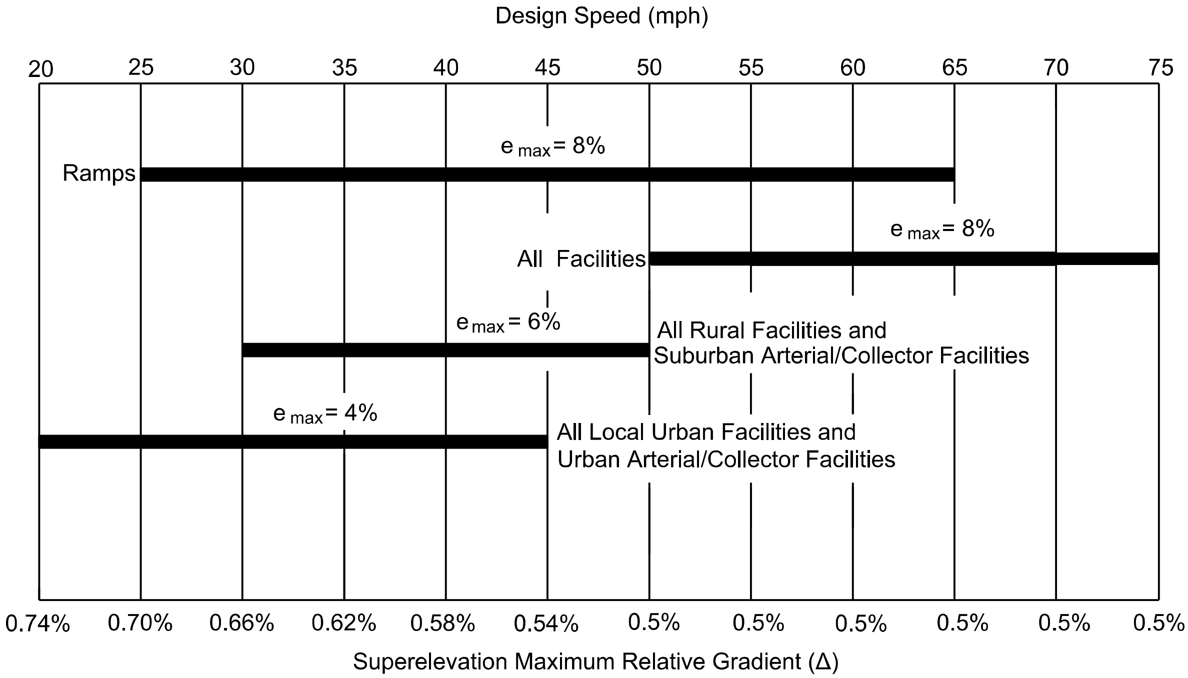
1. Open-Roadway Conditions. Superelevation and side friction are distributed by AASHTO Method 5, which allows e and f to gradually increase in a curvilinear manner up to e_{max} and f_{max}. This method yields superelevation rates for which the superelevation counteracts nearly all lateral acceleration at the average running speed and, therefore, considerable side friction is available for those drivers who are traveling near or above the design speed. Section 5.3.2 presents the superelevation rates that result from the use of Method 5.

2. Low-Speed Urban Streets. Superelevation and side friction are distributed by AASHTO Method 2, which allows f to increase up to f_{max} before any superelevation is introduced. The practical effect of AASHTO Method 2 is that superelevation is rarely warranted on low-speed urban streets ($V_d \leq 45$ mph). For this method of distribution, the superelevation rates may be calculated directly from Equation 5.2-1 using $f = f_{max}$. Section 5.3.3 presents the superelevation rates that result from the use of Method 2.

The distribution methodology for superelevation and side friction determines the development of superelevation criteria presented in Sections 5.3.2 and 5.3.3.

5.3.1.2 Maximum Superelevation Rate

As discussed in Section 5.2, the selection of a maximum rate of superelevation (e_{max}) depends upon several factors. These include design speed, urban/rural location, type of existing or expected roadside development, type of traffic operations expected and prevalent climatic conditions within South Carolina. For new construction/reconstruction projects, Figure 5.3-A summarizes the Department's selection of e_{max}.



APPLICATION OF e_{max} AND MAXIMUM RELATIVE GRADIENTS
Figure 5.3-A

5.3.2 Superelevation (Open-Roadway Conditions)

Open-roadway conditions are typically used on all rural highways and all urban facilities where V_d > 45 miles per hour. These types of facilities generally exhibit relatively uniform traffic operations. Therefore, for superelevation development, the flexibility normally exists to design horizontal curves with the more conservative AASHTO Method 5 (for distribution of superelevation and side friction).

Based on the selection of e_{max} and the use of AASHTO Method 5 to distribute e and f, the following figures allow the designer to select the design superelevation rate (e_d) for any combination of radius of curvature (R) and design speed (V_d):

1. Figure 5.3-B applies to e_{max} = 8 percent for V_d = 50 to 75 miles per hour.
2. Figure 5.3-C applies to e_{max} = 6 percent for V_d = 30 to 50 miles per hour.
3. Figure 5.3-D applies to e_{max} = 4 percent for V_d = 20 to 45 miles per hour.
4. Figure 5.3-E applies to e_{max} = 8 percent for interchange ramps with V_d = 25 to 65 miles per hour.

A horizontal curve with a sufficiently large radius does not require superelevation, and the normal crown section (NC) used on tangent can be maintained throughout the curve. This will apply for any radii greater than that shown in the first row of Figures 5.3-B, 5.3-C, 5.3-D and 5.3-E. On sharper curves for the same design speed, a point is reached where a uniform superelevation rate of 2.00 percent (the normal cross slope) is provided across the total traveled way (i.e., remove adverse crown (RC)). The radii noted in the second row (RC) in the figures are the curve radii where the RC applies. For sharper radii, superelevation rates steeper than 2.00 percent are necessary.

Super-elevation e_d	V_d = 50 mph		V_d = 55 mph		V_d = 60 mph		V_d = 65 mph	
	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)
NC	R ≥ 8150	-	R ≥ 9720	-	R ≥ 11500	-	R ≥ 12900	-
RC	5990 ≤ R < 8150	48	7150 ≤ R < 9720	48	8440 ≤ R < 11500	48	9510 ≤ R <12900	48
2.2%	5400 ≤ R < 5990	53	6450 ≤ R < 7150	53	7620 ≤ R < 8440	53	8600 ≤ R < 9510	53
2.4%	4910 ≤ R < 5400	58	5870 ≤ R < 6450	58	6930 ≤ R < 7620	58	7830 ≤ R < 8600	58
2.6%	4490 ≤ R < 4910	62	5370 ≤ R < 5870	62	6350 ≤ R < 6930	62	7180 ≤ R < 7830	62
2.8%	4130 ≤ R < 4490	67	4950 ≤ R < 5370	67	5850 ≤ R < 6350	67	6630 ≤ R < 7180	67
3.0%	3820 ≤ R < 4130	72	4580 ≤ R < 4950	72	5420 ≤ R < 5850	72	6140 ≤ R < 6630	72
3.2%	3550 ≤ R < 3820	77	4250 ≤ R < 4580	77	5040 ≤ R < 5420	77	5720 ≤ R < 6140	77
3.4%	3300 ≤ R < 3550	82	3970 ≤ R < 4250	82	4700 ≤ R < 5040	82	5350 ≤ R < 5720	82
3.6%	3090 ≤ R < 3300	86	3710 ≤ R < 3970	86	4400 ≤ R < 4700	86	5010 ≤ R < 5350	86
3.8%	2890 ≤ R < 3090	91	3480 ≤ R < 3710	91	4140 ≤ R < 4400	91	4710 ≤ R < 5010	91
4.0%	2720 ≤ R < 2890	96	3270 ≤ R < 3480	96	3890 ≤ R < 4140	96	4450 ≤ R < 4710	96
4.2%	2560 ≤ R < 2720	101	3080 ≤ R < 3270	101	3670 ≤ R < 3890	101	4200 ≤ R < 4450	101
4.4%	2410 ≤ R < 2560	106	2910 ≤ R < 3080	106	3470 ≤ R < 3670	106	3980 ≤ R < 4200	106
4.6%	2280 ≤ R < 2410	110	2750 ≤ R < 2910	110	3290 ≤ R < 3470	110	3770 ≤ R < 3980	110
4.8%	2160 ≤ R < 2280	115	2610 ≤ R < 2750	115	3120 ≤ R < 3290	115	3590 ≤ R < 3770	115
5.0%	2040 ≤ R < 2160	120	2470 ≤ R < 2610	120	2960 ≤ R < 3120	120	3410 ≤ R < 3590	120
5.2%	1930 ≤ R < 2040	125	2350 ≤ R < 2470	125	2820 ≤ R < 2960	125	3250 ≤ R < 3410	125
5.4%	1830 ≤ R < 1930	130	2230 ≤ R < 2350	130	2680 ≤ R < 2820	130	3110 ≤ R < 3250	130
5.6%	1740 ≤ R < 1830	134	2120 ≤ R < 2230	134	2550 ≤ R < 2680	134	2970 ≤ R < 3110	134
5.8%	1650 ≤ R < 1740	139	2010 ≤ R < 2120	139	2430 ≤ R < 2550	139	2840 ≤ R < 2970	139
6.0%	1560 ≤ R < 1650	144	1920 ≤ R < 2010	144	2320 ≤ R < 2430	144	2710 ≤ R < 2840	144
6.2%	1480 ≤ R < 1560	149	1820 ≤ R < 1920	149	2210 ≤ R < 2320	149	2600 ≤ R < 2710	149
6.4%	1400 ≤ R < 1480	154	1730 ≤ R < 1820	154	2110 ≤ R < 2210	154	2490 ≤ R < 2600	154
6.6%	1330 ≤ R < 1400	158	1650 ≤ R < 1730	158	2010 ≤ R < 2110	158	2380 ≤ R < 2490	158
6.8%	1260 ≤ R < 1330	163	1560 ≤ R < 1650	163	1910 ≤ R < 2010	163	2280 ≤ R < 2380	163
7.0%	1190 ≤ R < 1260	168	1480 ≤ R < 1560	168	1820 ≤ R < 1910	168	2180 ≤ R < 2280	168
7.2%	1120 ≤ R < 1190	173	1400 ≤ R < 1480	173	1720 ≤ R < 1820	173	2070 ≤ R < 2180	173
7.4%	1060 ≤ R < 1120	178	1320 ≤ R < 1400	178	1630 ≤ R < 1720	178	1970 ≤ R < 2070	178
7.6%	980 ≤ R < 1060	182	1230 ≤ R < 1320	182	1530 ≤ R < 1630	182	1850 ≤ R < 1970	182
7.8%	901 ≤ R < 980	187	1140 ≤ R < 1230	187	1410 ≤ R < 1530	187	1720 ≤ R < 1850	187
8.0%	758 ≤ R < 901	192	960 ≤ R < 1140	192	1200 ≤ R < 1410	192	1480 ≤ R < 1720	192

Key to Table:

R = *Radius of curve in feet.*

V_d = *Design speed in miles per hour.*

e_d = *Design superelevation rate shown as a percent.*

L_r = *Minimum length of superelevation runoff in feet from adverse cross slope removed (cross slope of outside lane is level) to full superelevation for a two-lane highway and the point of revolution about the centerline. Add the calculated tangent runout length to this number. Values in table assume 12-foot lane widths. Use Equation 5.3-1 for other lane widths.*

NC = *Normal crown (2.00 percent typical).*

RC = *Remove adverse crown (i.e., superelevate traveled way at normal cross slope) (2.00 percent typical).*

SUPERELEVATION RATE (e_d) (e_{max} = 8.0 Percent)
(All Facilities)
Figure 5.3-B

(continued on next page)

Superelevation e_d	V_d = 70 mph		V_d = 75 mph	
	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)
NC	R ≥ 14500	-	R ≥ 16100	-
RC	10700 ≤ R < 14500	48	12000 ≤ R < 16100	48
2.2%	9660 ≤ R < 10700	53	10800 ≤ R < 12000	53
2.4%	8810 ≤ R < 9660	58	9850 ≤ R < 10800	58
2.6%	8090 ≤ R < 8810	62	9050 ≤ R < 9850	62
2.8%	7470 ≤ R < 8090	67	8370 ≤ R < 9050	67
3.0%	6930 ≤ R < 7470	72	7780 ≤ R < 8370	72
3.2%	6460 ≤ R < 6930	77	7260 ≤ R < 7780	77
3.4%	6050 ≤ R < 6460	82	6800 ≤ R < 7260	82
3.6%	5680 ≤ R < 6050	86	6400 ≤ R < 6800	86
3.8%	5350 ≤ R < 5680	91	6030 ≤ R < 6400	91
4.0%	5050 ≤ R < 5350	96	5710 ≤ R < 6030	96
4.2%	4780 ≤ R < 5050	101	5410 ≤ R < 5710	101
4.4%	4540 ≤ R < 4780	106	5140 ≤ R < 5410	106
4.6%	4310 ≤ R < 4540	110	4890 ≤ R < 5140	110
4.8%	4100 ≤ R < 4310	115	4670 ≤ R < 4890	115
5.0%	3910 ≤ R < 4100	120	4460 ≤ R < 4670	120
5.2%	3740 ≤ R < 3910	125	4260 ≤ R < 4460	125
5.4%	3570 ≤ R < 3740	130	4090 ≤ R < 4260	130
5.6%	3420 ≤ R < 3570	134	3920 ≤ R < 4090	134
5.8%	3280 ≤ R < 3420	139	3760 ≤ R < 3920	139
6.0%	3150 ≤ R < 3280	144	3620 ≤ R < 3760	144
6.2%	3020 ≤ R < 3150	149	3480 ≤ R < 3620	149
6.4%	2910 ≤ R < 3020	154	3360 ≤ R < 3480	154
6.6%	2790 ≤ R < 2910	158	3240 ≤ R < 3360	158
6.8%	2690 ≤ R < 2790	163	3120 ≤ R < 3240	163
7.0%	2580 ≤ R < 2690	168	3010 ≤ R < 3120	168
7.2%	2470 ≤ R < 2580	173	2900 ≤ R < 3010	173
7.4%	2350 ≤ R < 2470	178	2780 ≤ R < 2900	178
7.6%	2230 ≤ R < 2350	182	2650 ≤ R < 2780	182
7.8%	2090 ≤ R < 2230	187	2500 ≤ R < 2650	187
8.0%	1810 ≤ R < 2090	192	2210 ≤ R < 2500	192

Key to Table:

R = *Radius of curve in feet.*

V_d = *Design speed in miles per hour.*

e_d = *Design superelevation rate shown as a percent.*

L_r = *Minimum length of superelevation runoff in feet from adverse cross slope removed (cross slope of outside lane is level) to full superelevation for a two-lane highway and the point of revolution about the centerline. Add the calculated tangent runout length to this number. Values in table assume 12-foot lane widths. Use Equation 5.3-1 for other lane widths.*

NC = *Normal crown (2.00 percent typical).*

RC = *Remove adverse crown (i.e., superelevate traveled way at normal cross slope) (2.00 percent typical).*

SUPERELEVATION RATE (e_d) (e_{max} = 8.0 Percent)
(All Facilities)
Figure 5.3-B
(Continued)

Superelevation e_d	V_d = 30 mph		V_d = 35 mph		V_d = 40 mph		V_d = 45 mph	
	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)
NC	R ≥ 3130	-	R ≥ 4100	-	R ≥ 5230	-	R ≥ 6480	-
RC	2240 ≤ R < 3130	36	2950 ≤ R < 4100	39	3770 ≤ R < 5230	41	4680 ≤ R < 6480	44
2.2%	2000 ≤ R < 2240	40	2630 ≤ R < 2950	43	3370 ≤ R < 3770	46	4190 ≤ R < 4680	49
2.4%	1790 ≤ R < 2000	44	2360 ≤ R < 2630	46	3030 ≤ R < 3370	50	3770 ≤ R < 4190	53
2.6%	1610 ≤ R < 1790	47	2130 ≤ R < 2360	50	2740 ≤ R < 3030	54	3420 ≤ R < 3770	58
2.8%	1460 ≤ R < 1610	51	1930 ≤ R < 2130	54	2490 ≤ R < 2740	58	3110 ≤ R < 3420	62
3.0%	1320 ≤ R < 1460	55	1760 ≤ R < 1930	58	2270 ≤ R < 2490	62	2840 ≤ R < 3110	67
3.2%	1200 ≤ R < 1320	58	1600 ≤ R < 1760	62	2080 ≤ R < 2270	66	2600 ≤ R < 2840	71
3.4%	1080 ≤ R < 1200	62	1460 ≤ R < 1600	66	1900 ≤ R < 2080	70	2390 ≤ R < 2600	76
3.6%	972 ≤ R < 1080	65	1320 ≤ R < 1460	70	1740 ≤ R < 1900	74	2190 ≤ R < 2390	80
3.8%	864 ≤ R < 972	69	1190 ≤ R < 1320	74	1590 ≤ R < 1740	79	2010 ≤ R < 2190	84
4.0%	766 ≤ R < 864	73	1070 ≤ R < 1190	77	1440 ≤ R < 1590	83	1840 ≤ R < 2010	89
4.2%	684 ≤ R < 766	76	960 ≤ R < 1070	81	1310 ≤ R < 1440	87	1680 ≤ R < 1840	93
4.4%	615 ≤ R < 684	80	868 ≤ R < 960	85	1190 ≤ R < 1310	91	1540 ≤ R < 1680	98
4.6%	555 ≤ R < 615	84	788 ≤ R < 868	89	1090 ≤ R < 1190	95	1410 ≤ R < 1540	102
4.8%	502 ≤ R < 555	87	718 ≤ R < 788	93	995 ≤ R < 1090	99	1300 ≤ R < 1410	107
5.0%	456 ≤ R < 502	91	654 ≤ R < 718	97	911 ≤ R < 995	103	1190 ≤ R < 1300	111
5.2%	413 ≤ R < 456	95	595 ≤ R < 654	101	833 ≤ R < 911	108	1090 ≤ R < 1190	116
5.4%	373 ≤ R < 413	98	540 ≤ R < 595	105	759 ≤ R < 833	112	995 ≤ R < 1090	120
5.6%	335 ≤ R < 373	102	487 ≤ R < 540	108	687 ≤ R < 759	116	903 ≤ R < 995	124
5.8%	296 ≤ R < 335	105	431 ≤ R < 487	112	611 ≤ R < 687	120	806 ≤ R < 903	129
6.0%	231 ≤ R < 296	109	340 ≤ R < 431	116	485 ≤ R < 611	124	643 ≤ R < 806	133

Superelevation e_d	V_d = 50 mph	
	Radius (ft)	L_r (ft)
NC	R ≥ 7870	-
RC	5700 ≤ R < 7870	48
2.2%	5100 ≤ R < 5700	53
2.4%	4600 ≤ R < 5100	58
2.6%	4170 ≤ R < 4600	62
2.8%	3800 ≤ R < 4170	67
3.0%	3480 ≤ R < 3800	72
3.2%	3200 ≤ R < 3480	77
3.4%	2940 ≤ R < 3200	82
3.6%	2710 ≤ R < 2940	86
3.8%	2490 ≤ R < 2710	91
4.0%	2300 ≤ R < 2490	96
4.2%	2110 ≤ R < 2300	101
4.4%	1940 ≤ R < 2110	106
4.6%	1780 ≤ R < 1940	110
4.8%	1640 ≤ R < 1780	115
5.0%	1510 ≤ R < 1640	120
5.2%	1390 ≤ R < 1510	125
5.4%	1280 ≤ R < 1390	130
5.6%	1160 ≤ R < 1280	134
5.8%	1040 ≤ R < 1160	139
6.0%	833 ≤ R < 1040	144

Key to Table:

R = Radius of curve in feet.

V_d = Design speed in miles per hour.

e_d = Design superelevation rate shown as a percent.

L_r = Minimum length of superelevation runoff in feet from adverse cross slope removed (cross slope of outside lane is level) to full superelevation for a two-lane highway and the point of revolution about the centerline. Add the calculated tangent runout length to this number. Values in table assume 12-foot lane widths. Use Equation 5.3-1 for other lane widths.

NC = Normal crown (2.00 percent typical).

RC = Remove adverse crown (i.e., superelevate traveled way at normal cross slope) (2.00 percent typical).

SUPERELEVATION RATE (e_d) (e_{max} = 6.0 Percent)
Figure 5.3-C

Superelevation e_d	V_d = 20 mph		V_d = 25 mph		V_d = 30 mph		V_d = 35 mph	
	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)
NC	R ≥ 1410	-	R ≥ 2050	-	R ≥ 2830	-	R ≥ 3730	-
RC	902 ≤ R < 1410	32	1340 ≤ R < 2050	34	1880 ≤ R < 2830	36	2490 ≤ R < 3730	39
2.2%	723 ≤ R < 902	36	1110 ≤ R < 1340	38	1580 ≤ R < 1880	40	2120 ≤ R < 2490	43
2.4%	513 ≤ R < 723	39	838 ≤ R < 1110	41	1270 ≤ R < 1580	44	1760 ≤ R < 2120	46
2.6%	388 ≤ R < 513	42	650 ≤ R < 838	45	1000 ≤ R < 1270	47	1420 ≤ R < 1760	50
2.8%	308 ≤ R < 388	45	524 ≤ R < 650	48	817 ≤ R < 1000	51	1170 ≤ R < 1420	54
3.0%	251 ≤ R < 308	49	433 ≤ R < 524	51	681 ≤ R < 817	55	982 ≤ R < 1170	58
3.2%	209 ≤ R < 251	52	363 ≤ R < 433	55	576 ≤ R < 681	58	835 ≤ R < 982	62
3.4%	175 ≤ R < 209	55	307 ≤ R < 363	58	490 ≤ R < 576	62	714 ≤ R < 835	66
3.6%	147 ≤ R < 175	58	259 ≤ R < 307	62	416 ≤ R < 490	65	610 ≤ R < 714	70
3.8%	122 ≤ R < 147	62	215 ≤ R < 259	65	348 ≤ R < 416	69	512 ≤ R < 610	74
4.0%	86 ≤ R < 122	65	154 ≤ R < 215	69	250 ≤ R < 348	73	371 ≤ R < 512	77

Superelevation e_d	V_d = 40 mph		V_d = 45 mph	
	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)
NC	R ≥ 4770	-	R ≥ 5930	-
RC	3220 ≤ R < 4770	41	4040 ≤ R < 5930	44
2.2%	2760 ≤ R < 3220	46	3480 ≤ R < 4040	49
2.4%	2340 ≤ R < 2760	50	2980 ≤ R < 3480	53
2.6%	1930 ≤ R < 2340	54	2490 ≤ R < 2980	58
2.8%	1620 ≤ R < 1930	58	2100 ≤ R < 2490	62
3.0%	1370 ≤ R < 1620	62	1800 ≤ R < 2100	67
3.2%	1180 ≤ R < 1370	66	1550 ≤ R < 1800	71
3.4%	1010 ≤ R < 1180	70	1340 ≤ R < 1550	76
3.6%	865 ≤ R < 1010	74	1150 ≤ R < 1340	80
3.8%	730 ≤ R < 865	79	970 ≤ R < 1150	84
4.0%	533 ≤ R < 730	83	711 ≤ R < 970	89

Key to Table:

R = *Radius of curve in feet.*

V_d = *Design speed in miles per hour.*

e_d = *Design superelevation rate shown as a percent.*

L_r = *Minimum length of superelevation runoff in feet from adverse cross slope removed (cross slope of outside lane is level) to full superelevation for a two-lane highway and the point of revolution about the centerline. Add the calculated tangent runout length to this number. Values in table assume 12-foot lane widths. Use Equation 5.3-1 for other lane widths.*

NC = *Normal crown (2.00 percent typical).*

RC = *Remove adverse crown (i.e., superelevate traveled way at normal cross slope) (2.00 percent typical).*

SUPERELEVATION RATE (e_d) (e_{max} = 4.0 Percent)
Figure 5.3-D

Superelevation e_d	V_d = 25 mph		V_d = 30 mph		V_d = 35 mph		V_d = 40 mph		V_d = 45 mph	
	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)
NC	R ≥ 2370	-	R ≥ 3240	-	R ≥ 4260	-	R ≥ 5410	-	R ≥ 6710	-
RC	1720 ≤ R < 2370	46	2370 ≤ R < 3240	48	3120 ≤ R < 4260	52	3970 ≤ R < 5410	55	4930 ≤ R < 6710	59
2.2%	1550 ≤ R < 1720	50	2130 ≤ R < 2370	53	2800 ≤ R < 3120	57	3570 ≤ R < 3970	61	4440 ≤ R < 4930	65
2.4%	1400 ≤ R < 1550	55	1930 ≤ R < 2130	58	2540 ≤ R < 2800	62	3240 ≤ R < 3570	66	4030 ≤ R < 4440	71
2.6%	1280 ≤ R < 1400	59	1760 ≤ R < 1930	63	2320 ≤ R < 2540	67	2960 ≤ R < 3240	72	3690 ≤ R < 4030	77
2.8%	1170 ≤ R < 1280	64	1610 ≤ R < 1760	68	2130 ≤ R < 2320	72	2720 ≤ R < 2960	77	3390 ≤ R < 3690	83
3.0%	1070 ≤ R < 1170	69	1480 ≤ R < 1610	73	1960 ≤ R < 2130	77	2510 ≤ R < 2720	83	3130 ≤ R < 3390	89
3.2%	985 ≤ R < 1070	73	1370 ≤ R < 1480	78	1820 ≤ R < 1960	83	2330 ≤ R < 2510	88	2900 ≤ R < 3130	95
3.4%	911 ≤ R < 985	78	1270 ≤ R < 1370	82	1690 ≤ R < 1820	88	2170 ≤ R < 2330	94	2700 ≤ R < 2900	101
3.6%	845 ≤ R < 911	82	1180 ≤ R < 1270	87	1570 ≤ R < 1690	93	2020 ≤ R < 2170	99	2520 ≤ R < 2700	107
3.8%	784 ≤ R < 845	87	1100 ≤ R < 1180	92	1470 ≤ R < 1570	98	1890 ≤ R < 2020	105	2360 ≤ R < 2520	113
4.0%	729 ≤ R < 784	91	1030 ≤ R < 1100	97	1370 ≤ R < 1470	103	1770 ≤ R < 1890	110	2220 ≤ R < 2360	119
4.2%	678 ≤ R < 729	96	955 ≤ R < 1030	102	1280 ≤ R < 1370	108	1660 ≤ R < 1770	116	2080 ≤ R < 2220	124
4.4%	630 ≤ R < 678	101	893 ≤ R < 955	107	1200 ≤ R < 1280	114	1560 ≤ R < 1660	121	1960 ≤ R < 2080	130
4.6%	585 ≤ R < 630	105	834 ≤ R < 893	112	1130 ≤ R < 1200	119	1470 ≤ R < 1560	127	1850 ≤ R < 1960	136
4.8%	542 ≤ R < 585	110	779 ≤ R < 834	116	1060 ≤ R < 1130	124	1390 ≤ R < 1470	132	1750 ≤ R < 1850	142
5.0%	499 ≤ R < 542	114	727 ≤ R < 779	121	991 ≤ R < 1060	129	1310 ≤ R < 1390	138	1650 ≤ R < 1750	148
5.2%	457 ≤ R < 499	119	676 ≤ R < 727	126	929 ≤ R < 991	134	1230 ≤ R < 1310	143	1560 ≤ R < 1650	154
5.4%	420 ≤ R < 457	123	627 ≤ R < 676	131	870 ≤ R < 929	139	1160 ≤ R < 1230	149	1480 ≤ R < 1560	160
5.6%	387 ≤ R < 420	128	582 ≤ R < 627	136	813 ≤ R < 870	145	1090 ≤ R < 1160	154	1390 ≤ R < 1480	166
5.8%	358 ≤ R < 387	133	542 ≤ R < 582	141	761 ≤ R < 813	150	1030 ≤ R < 1090	160	1320 ≤ R < 1390	172
6.0%	332 ≤ R < 358	137	506 ≤ R < 542	145	713 ≤ R < 761	155	965 ≤ R < 1030	166	1250 ≤ R < 1320	178
6.2%	308 ≤ R < 332	142	472 ≤ R < 506	150	669 ≤ R < 713	160	909 ≤ R < 965	171	1180 ≤ R < 1250	184
6.4%	287 ≤ R < 308	146	442 ≤ R < 472	155	628 ≤ R < 669	165	857 ≤ R < 909	177	1110 ≤ R < 1180	190
6.6%	267 ≤ R < 287	151	413 ≤ R < 442	160	590 ≤ R < 628	170	808 ≤ R < 857	182	1050 ≤ R < 1110	196
6.8%	248 ≤ R < 267	155	386 ≤ R < 413	165	553 ≤ R < 590	175	761 ≤ R < 808	188	990 ≤ R < 1050	201
7.0%	231 ≤ R < 248	160	360 ≤ R < 386	170	518 ≤ R < 553	181	716 ≤ R < 761	193	933 ≤ R < 990	207
7.2%	214 ≤ R < 231	165	336 ≤ R < 360	175	485 ≤ R < 518	186	672 ≤ R < 716	199	878 ≤ R < 933	213
7.4%	198 ≤ R < 214	169	312 ≤ R < 336	179	451 ≤ R < 485	191	628 ≤ R < 672	204	822 ≤ R < 878	219
7.6%	182 ≤ R < 198	174	287 ≤ R < 312	184	417 ≤ R < 451	196	583 ≤ R < 628	210	765 ≤ R < 822	225
7.8%	164 ≤ R < 182	178	261 ≤ R < 287	189	380 ≤ R < 417	201	533 ≤ R < 583	215	701 ≤ R < 765	231
8.0%	134 ≤ R < 164	183	214 ≤ R < 261	194	314 ≤ R < 380	206	444 ≤ R < 533	221	587 ≤ R < 701	237

Key to Table:

R = *Radius of curve in feet.*

V_d = *Design speed in miles per hour.*

e_d = *Design superelevation rate shown as a percent.*

L_r = *Minimum length of superelevation runoff, in feet, from adverse cross slope removed to full superelevation for a ramp. The point of revolution is about the edge of the traveled way. Values in table assume 16-foot ramp widths. Add the calculated tangent runout length to this value. Use Equation 5.3-1 for other ramp widths.*

NC = *Normal crown (2.00 percent typical).*

RC = *Remove adverse crown (i.e., superelevate traveled way at normal cross slope) (2.00 percent typical).*

SUPERELEVATION RATE (e_d) (e_{max} = 8.0 Percent)
(Interchange Ramps)
Figure 5.3-E
(continued on next page)

Superelevation e_d	V_d = 50 mph		V_d = 55 mph		V_d = 60 mph		V_d = 65 mph	
	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)	Radius (ft)	L_r (ft)
NC	R ≥ 8150	-	R ≥ 9720	-	R ≥ 11500	-	R ≥ 12900	-
RC	5990 ≤ R < 8150	64	7150 ≤ R < 9720	64	8440 ≤ R < 11500	64	9510 ≤ R <12900	64
2.2%	5400 ≤ R < 5990	70	6450 ≤ R < 7150	70	7620 ≤ R < 8440	70	8600 ≤ R < 9510	70
2.4%	4910 ≤ R < 5400	77	5870 ≤ R < 6450	77	6930 ≤ R < 7620	77	7830 ≤ R < 8600	77
2.6%	4490 ≤ R < 4910	83	5370 ≤ R < 5870	83	6350 ≤ R < 6930	83	7180 ≤ R < 7830	83
2.8%	4130 ≤ R < 4490	90	4950 ≤ R < 5370	90	5850 ≤ R < 6350	90	6630 ≤ R < 7180	90
3.0%	3820 ≤ R < 4130	96	4580 ≤ R < 4950	96	5420 ≤ R < 5850	96	6140 ≤ R < 6630	96
3.2%	3550 ≤ R < 3820	102	4250 ≤ R < 4580	102	5040 ≤ R < 5420	102	5720 ≤ R < 6140	102
3.4%	3300 ≤ R < 3550	109	3970 ≤ R < 4250	109	4700 ≤ R < 5040	109	5350 ≤ R < 5720	109
3.6%	3090 ≤ R < 3300	115	3710 ≤ R < 3970	115	4400 ≤ R < 4700	115	5010 ≤ R < 5350	115
3.8%	2890 ≤ R < 3090	122	3480 ≤ R < 3710	122	4140 ≤ R < 4400	122	4710 ≤ R < 5010	122
4.0%	2720 ≤ R < 2890	128	3270 ≤ R < 3480	128	3890 ≤ R < 4140	128	4450 ≤ R < 4710	128
4.2%	2560 ≤ R < 2720	134	3080 ≤ R < 3270	134	3670 ≤ R < 3890	134	4200 ≤ R < 4450	134
4.4%	2410 ≤ R < 2560	141	2910 ≤ R < 3080	141	3470 ≤ R < 3670	141	3980 ≤ R < 4200	141
4.6%	2280 ≤ R < 2410	147	2750 ≤ R < 2910	147	3290 ≤ R < 3470	147	3770 ≤ R < 3980	147
4.8%	2160 ≤ R < 2280	154	2610 ≤ R < 2750	154	3120 ≤ R < 3290	154	3590 ≤ R < 3770	154
5.0%	2040 ≤ R < 2160	160	2470 ≤ R < 2610	160	2960 ≤ R < 3120	160	3410 ≤ R < 3590	160
5.2%	1930 ≤ R < 2040	166	2350 ≤ R < 2470	166	2820 ≤ R < 2960	166	3250 ≤ R < 3410	166
5.4%	1830 ≤ R < 1930	173	2230 ≤ R < 2350	173	2680 ≤ R < 2820	173	3110 ≤ R < 3250	173
5.6%	1740 ≤ R < 1830	179	2120 ≤ R < 2230	179	2550 ≤ R < 2680	179	2970 ≤ R < 3110	179
5.8%	1650 ≤ R < 1740	186	2010 ≤ R < 2120	186	2430 ≤ R < 2550	186	2840 ≤ R < 2970	186
6.0%	1560 ≤ R < 1650	192	1920 ≤ R < 2010	192	2320 ≤ R < 2430	192	2710 ≤ R < 2840	192
6.2%	1480 ≤ R < 1560	198	1820 ≤ R < 1920	198	2210 ≤ R < 2320	198	2600 ≤ R < 2710	198
6.4%	1400 ≤ R < 1480	205	1730 ≤ R < 1820	205	2110 ≤ R < 2210	205	2490 ≤ R < 2600	205
6.6%	1330 ≤ R < 1400	211	1650 ≤ R < 1730	211	2010 ≤ R < 2110	211	2380 ≤ R < 2490	211
6.8%	1260 ≤ R < 1330	218	1560 ≤ R < 1650	218	1910 ≤ R < 2010	218	2280 ≤ R < 2380	218
7.0%	1190 ≤ R < 1260	224	1480 ≤ R < 1560	224	1820 ≤ R < 1910	224	2180 ≤ R < 2280	224
7.2%	1120 ≤ R < 1190	230	1400 ≤ R < 1480	230	1720 ≤ R < 1820	230	2070 ≤ R < 2180	230
7.4%	1060 ≤ R < 1120	237	1320 ≤ R < 1400	237	1630 ≤ R < 1720	237	1970 ≤ R < 2070	237
7.6%	980 ≤ R < 1060	243	1230 ≤ R < 1320	243	1530 ≤ R < 1630	243	1850 ≤ R < 1970	243
7.8%	901 ≤ R < 980	250	1140 ≤ R < 1230	250	1410 ≤ R < 1530	250	1720 ≤ R < 1850	250
8.0%	758 ≤ R < 901	256	960 ≤ R < 1140	256	1200 ≤ R < 1410	256	1480 ≤ R < 1720	256

Key to Table:

R = *Radius of curve in feet.*

V_d = *Design speed in miles per hour.*

e_d = *Design superelevation rate shown as a percent.*

L_r = *Minimum length of superelevation runoff, in feet, from adverse cross slope removed to full superelevation for a ramp. The point of revolution is about the edge of the traveled way. Values in table assume 16-foot ramp widths. Add the calculated tangent runout length to this value. Use Equation 5.3-1 for other ramps widths.*

NC = *Normal crown (2.00 percent typical).*

RC = *Remove adverse crown (i.e., superelevate traveled way at normal cross slope) (2.00 percent typical).*

SUPERELEVATION RATE (e_d) (e_{max} = 8.0 Percent)
(Interchange Ramps)
Figure 5.3-E
(Continued)

5.3.3 Superelevation (Low-Speed Urban Streets)

Although superelevation is beneficial for traffic operations, various factors often combine to make its use impractical in low-speed urban areas. These factors include:

- wide pavement areas;
- the need to meet the grade of adjacent property;
- surface drainage considerations;
- the desire to maintain low-speed operations; and
- frequency of intersecting cross streets, alleys and driveways.

Therefore, horizontal curves on low-speed urban streets are frequently designed without superelevation sustaining the lateral force solely with side friction. For vehicles on the outside of the curve, the normal cross slope (2.00 percent) results in a negative or adverse superelevation.

Where superelevation is required on low-speed urban streets (V_d ≤ 45 miles per hour), AASHTO Method 2 is used for determining the design superelevation. Figure 5.3-F provides the minimum radii and superelevation for low-speed urban streets.

e (%)	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph
	Radius (ft)	Radius (ft)	Radius (ft)	Radius (ft)
NC	R ≥ 107	R ≥ 198	R ≥ 333	R ≥ 510
-1.5%	105 ≤ R < 107	194 ≤ R < 198	324 ≤ R < 333	495 ≤ R < 510
0%	99 ≤ R < 105	181 ≤ R < 194	300 ≤ R < 324	454 ≤ R < 495
1.5%	94 ≤ R < 99	170 ≤ R < 181	279 ≤ R < 300	419 ≤ R < 454
2.0%	92 ≤ R < 94	167 ≤ R < 170	273 ≤ R < 279	408 ≤ R < 419
2.2%	91 ≤ R < 92	165 ≤ R < 167	270 ≤ R < 273	404 ≤ R < 408
2.4%	91 ≤ R < 91	164 ≤ R < 165	268 ≤ R < 270	400 ≤ R < 404
2.6%	90 ≤ R < 91	163 ≤ R < 164	265 ≤ R < 268	396 ≤ R < 400
2.8%	89 ≤ R < 90	161 ≤ R < 163	263 ≤ R < 265	393 ≤ R < 396
3.0%	89 ≤ R < 89	160 ≤ R < 161	261 ≤ R < 263	389 ≤ R < 393
3.2%	88 ≤ R < 89	159 ≤ R < 160	259 ≤ R < 261	385 ≤ R < 389
3.4%	88 ≤ R < 88	158 ≤ R < 159	256 ≤ R < 259	382 ≤ R < 385
3.6%	87 ≤ R < 88	157 ≤ R < 158	254 ≤ R < 256	378 ≤ R < 382
3.8%	87 ≤ R < 87	155 ≤ R < 157	252 ≤ R < 254	375 ≤ R < 378
4.0%	86 ≤ R < 87	154 ≤ R < 155	250 ≤ R < 252	371 ≤ R < 375

e (%)	V_d = 40 mph	V_d = 45 mph
	Radius (ft)	Radius (ft)
NC	R ≥ 762	R ≥ 1039
-1.5%	736 ≤ R < 762	1000 ≤ R < 1039
0%	667 ≤ R < 736	900 ≤ R < 1000
1.5%	610 ≤ R < 667	818 ≤ R < 900
2.0%	593 ≤ R < 610	794 ≤ R < 818
2.2%	586 ≤ R < 593	785 ≤ R < 794
2.4%	580 ≤ R < 586	776 ≤ R < 785
2.6%	573 ≤ R < 580	767 ≤ R < 776
2.8%	567 ≤ R < 573	758 ≤ R < 767
3.0%	561 ≤ R < 567	750 ≤ R < 758
3.2%	556 ≤ R < 561	742 ≤ R < 750
3.4%	550 ≤ R < 556	734 ≤ R < 742
3.6%	544 ≤ R < 550	726 ≤ R < 734
3.8%	539 ≤ R < 544	718 ≤ R < 726
4.0%	533 ≤ R < 539	711 ≤ R < 718

Key to Table:

R = Radius of curve in feet.

V_d = Design speed in miles per hour.

e_d = Design superelevation rate shown as a percent.

NC = Normal crown (2.00 percent typical).

RC = Remove adverse crown (i.e., superelevate traveled way at normal cross slope) (2.00 percent typical).

MINIMUM RADII AND SUPERELEVATION FOR LOW-SPEED URBAN STREETS
Figure 5.3-F

5.3.4 Superelevation Transition Lengths

As defined in Section 5.1, the superelevation transition length is the distance required to transition the roadway from a normal crown section to the full design superelevation rate. The superelevation transition length is the sum of the tangent runout distance (L_t) and superelevation runoff length (L_r).

Several highway features may significantly influence superelevation development for multilane highways. These include guardrail, median barriers, drainage and major at-grade intersections. The designer should carefully consider the intended function of all highway features and ensure that the superelevated section and selected point of revolution does not compromise traffic operations. In addition, the designer should consider the likely ultimate development of the facility and select a point of revolution that will lend itself to future expansion.

5.3.4.1 Point of Revolution

The following discusses typical locations for points of revolution and how to determine the superelevation transition length:

1. Superelevation Runoff. The e_{max} tables (Figures 5.3-B through 5.3-D) present the superelevation runoff lengths (L_r) for two-lane highways. Superelevation runoff lengths for two-lane and multilane facilities are calculated as follows:

$$L_r = \frac{(w\, n_1)\, e_d}{\Delta}(b_w) \qquad \text{(Equation 5.3-1)}$$

Where:

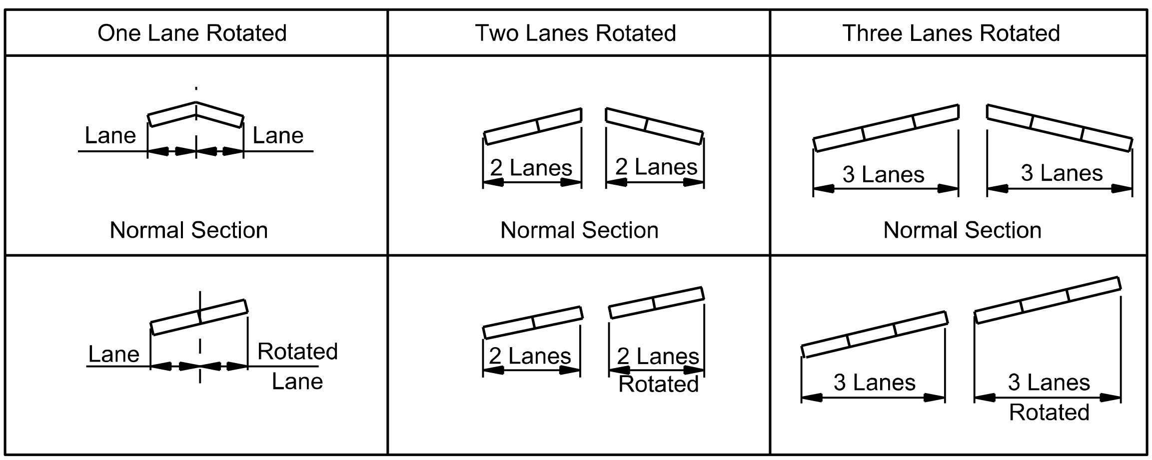
L_r	=	Minimum superelevation runoff length (assuming the point of revolution is about the roadway centerline), feet
e_d	=	Design superelevation rate, decimal
n_1	=	Number of lanes rotated
b_w	=	Adjustment factor for number of lanes rotated (see Figure 5.3-G)
w	=	Width of rotation (assumed to be 12 feet), feet
Δ	=	Maximum relative gradient between the profile grade and the edge of adjacent travel lane, decimal (see Figure 5.3-A)

a. Two-Lane Highway. The L_r values in Figures 5.3-B through 5.3-D are applicable for two-lane highways where the width (w) of the pavement rotated is 12 feet; n_1 and b_w are each 1. The point of revolution will typically be about the centerline of the roadway; see Figure 5.3-H(a) and (b). This will yield the least amount of elevation differential between the pavement edges and their normal profiles. Occasionally, it may be necessary to revolve about the inside or outside edge of the traveled way. This may be necessary to meet field conditions (e.g., drainage on a curbed facility, roadside development). Note that revolution about the edge of traveled way will require an increase in the superelevation runoff and tangent runout lengths as discussed in Item 1.d. for divided highways with narrow medians.

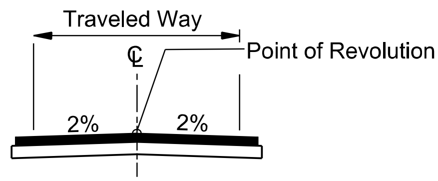
On a two-lane highway with an outer auxiliary lane (e.g., a climbing lane), the point of revolution will typically be about the centerline of the two through lanes.

Number of Lanes Rotated (n_1)	**Adjust Factor (b_w)***	**Length Increase Relative to One-Lane Rotated (n_1 x b_w)**
1	1.00	1.00
1.5	0.83	1.25
2	0.75	1.50
2.5	0.70	1.75
3	0.67	2.00
3.5	0.64	2.25
4	0.625	2.50

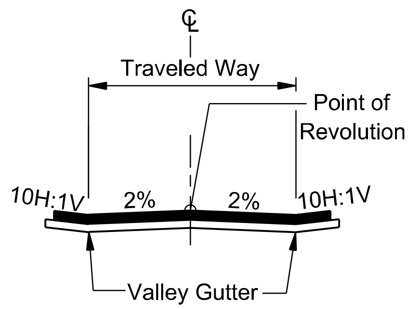
*$b_w = (1 + 0.5(n_1-1))/n_1$



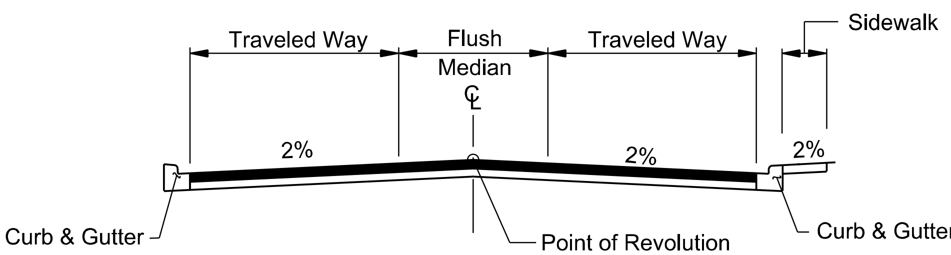
ADJUSTMENT FACTORS FOR THE NUMBER OF LANES ROTATED
Figure 5.3-G



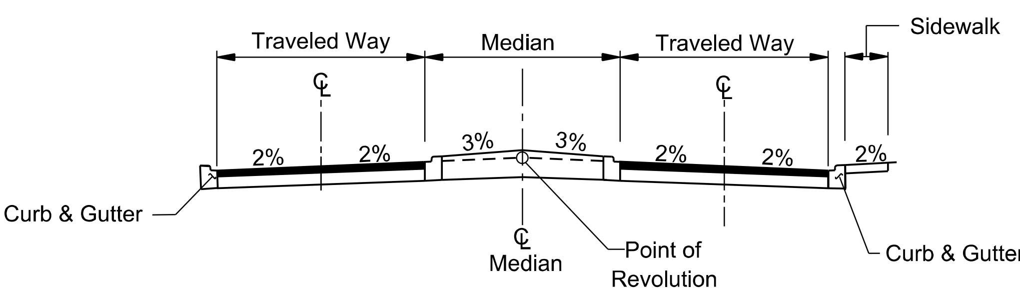
(a) Two-Lane Highway



(b) Two-Lane (Valley Gutter)

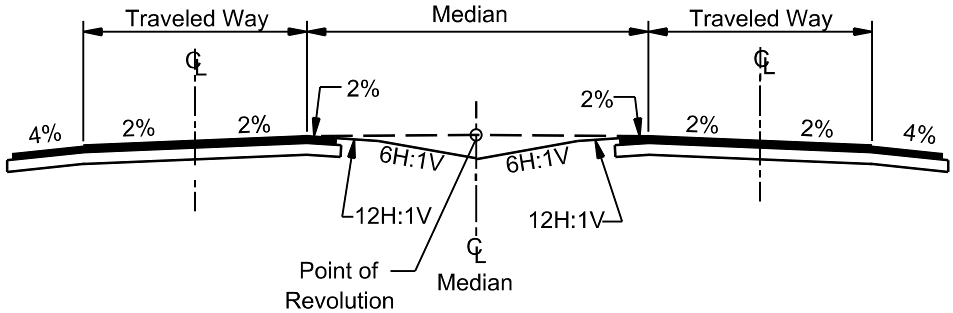


(c) Flush Median/TWLTL

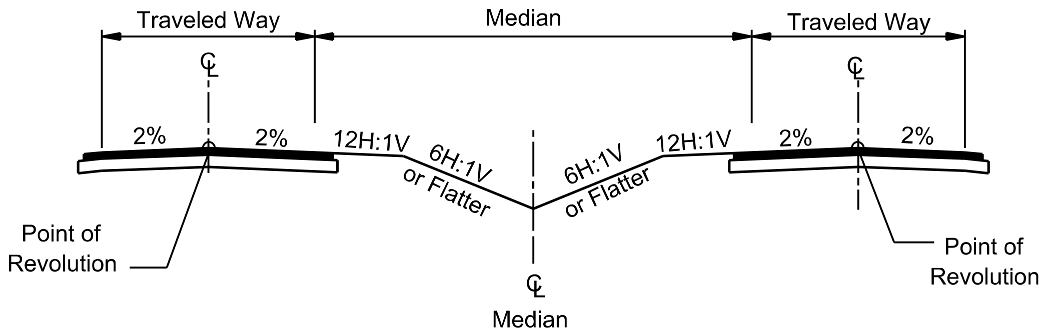


(d) Raised Median

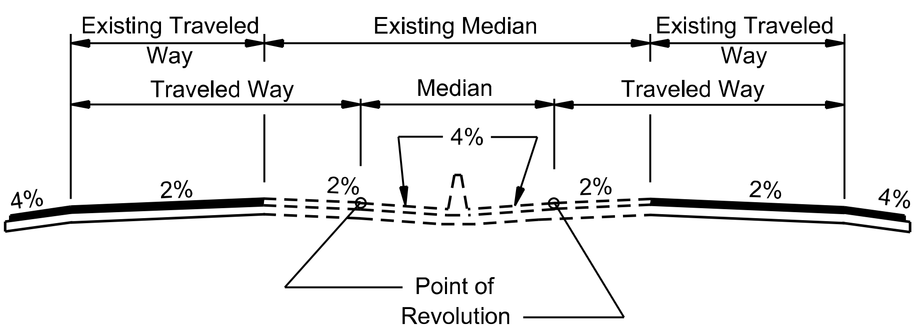
TYPICAL POINTS OF REVOLUTION
Figure 5.3-H



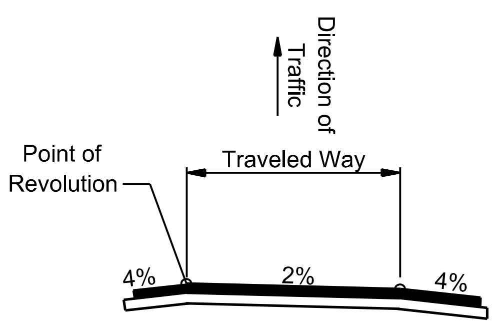
(e) Four-Lane Divided Highway
(Narrow Median)



(f) Four-Lane Divided Highway
(Wide Median)



(g) Future Travel Lanes



(h) One-Way Freeway Ramp

TYPICAL POINTS OF REVOLUTION
Figure 5.3-H
(Continued)

b. Undivided Multi-lane, Flush Median or TWLTL. For a typical urban undivided multilane highway with or without a flush median or two-way, left-turn lane (i.e., the crown is in the center of the roadway), the point of revolution is at the center of the roadway; see Figure 5.3-H(c). For the superelevation transition, the designer needs to determine b_w based on the number of lanes rotated. For example, a five-lane section with 12-foot travel lanes and a 15-foot TWLTL would be calculated as follows:

- First, determine the number of lanes rotated.

  n_1 = (12 ft + 12 ft + 15 ft/2) / 12 ft = 2.625 lanes

- Next, the designer needs to determine b_w. Because of the number of lanes rotate does not match the values shown in Figure 5.3-G, use the equation in Figure 5.3-G to determine b_w.

  $b_w = (1 + 0.5(n_1\text{-}1))/n_1 = (1 + 0.5\ (2.625 - 1))\ /\ 2.625 = 0.69$

- Use the above calculated values in Equation 5.3-1 to determine L_r.

c. Raised Medians. For divided highways with raised medians, the point of revolution will typically be the centerline of the entire roadway section; see Figure 5.3-H(d). The number of lanes (n_1) and the adjustment factor (b_w) are determined from the center of roadway section to the outside edge of traveled way in the same manner as discussed in Item 1.b., which will also include ½ the median width.

d. Divided Highways with Earthen, Depressed Medians. The most appropriate location for the point of revolution depends on the width of the median, the cross-section of the roadway and the potential to construct future travel lanes within the median. Refer to the AASHTO *A Policy on Geometric Design of Highways and Streets* for guidance on selecting the appropriate point of revolution.

e. Ramps. The L_r values in Figure 5.3-E are applicable for interchange ramps where the width (w) of the pavement rotated is 16 feet. The point of revolution will typically be at the inside edge of traveled way of the ramp; see Figure 5.3-H(h).

2. Tangent Runout. Calculate the tangent runout distance using the following equation:

$$L_t = \frac{e_{NC}}{e_d}(L_r) \qquad \text{(Equation 5.3-2)}$$

Where:

L_t = Minimum tangent runout length (assuming the point of revolution is about the roadway centerline), feet
e_{NC} = Normal cross slope (typically 0.0200), decimal
L_r = Superelevation runoff length, feet (See Equation 5.3-1)
e_d = Design superelevation rate, decimal

3. Superelevation Transition Length. Once the tangent runout (L_t) distance is calculated, add this distance to the design superelevation runoff length (L_r). The total of these two numbers equals the theoretical superelevation transition length used for design.

5.3.4.2 Application of Superelevation Transition Length

Once the superelevation transition length has been calculated, the designer must determine how to fit the length into the horizontal and vertical planes. The following will apply:

1. Simple Curves. Typically for new construction/reconstruction projects, 67 percent of the superelevation runoff length will be placed on the tangent and 33 percent on the curve. Exceptions to this practice may be necessary to meet field conditions. The generally accepted range is 60 percent to 80 percent on the tangent and 40 percent to 20 percent on the curve. In extreme cases, the superelevation runoff may be distributed 50 percent to 100 percent on the tangent and 50 percent to 0 percent on the curve. This will usually occur only in urban or suburban areas with highly restricted right-of-way conditions. When considering the tangent runout distance, this results in a distribution of the total superelevation transition length of approximately 75 percent on the tangent and 25 percent on the curve. However, because the distribution of the superelevation transition length is not an exact science, the ratio should be rounded slightly (e.g., to the nearest 5-foot increment) to simplify design and layout in construction.

2. Field Application (Vertical Profile). At the beginning and end of the superelevation transition length, angular breaks occur in the profile at the edge of the pavement if not smoothed. As a guide to eliminate angular breaks, the vertical curve transitions should have a length in feet numerically equivalent to the approximate design speed in miles per hour ±10 feet with 40 feet as a minimum. In addition, designers should graphically or numerically investigate the transition areas to identify potential flat spots for drainage before finalizing construction plans. When the edge of pavement profile is adjusted, the cross slopes and elevations on the cross sections should reflect these changes.

3. Ultimate Development. If the proposed facility is planned for an ultimate development of additional lanes, the designer should, where practical, reflect this length in the initial superelevation transition application. For example, a four-lane divided facility may be planned for an ultimate six-lane divided facility. Therefore, the superelevation transition

length for the initial four-lane facility should be consistent with the future requirements of the six-lane facility; see Figure 5.3-H(g).

5.3.5 Shoulder Treatment on Superelevated Curves

Figure 5.3-I illustrates the shoulder treatment for superelevated sections. The designer should note the following:

1. Low Side. For paved shoulders on new construction and reconstruction projects, retain the low-side shoulder slope until the adjacent superelevated travel lane reaches the normal shoulder slope. The shoulder is then superelevated concurrently with the travel lane until the design superelevation (e_d) is reached (i.e., the insider shoulder and travel lane will remain in a plane section).

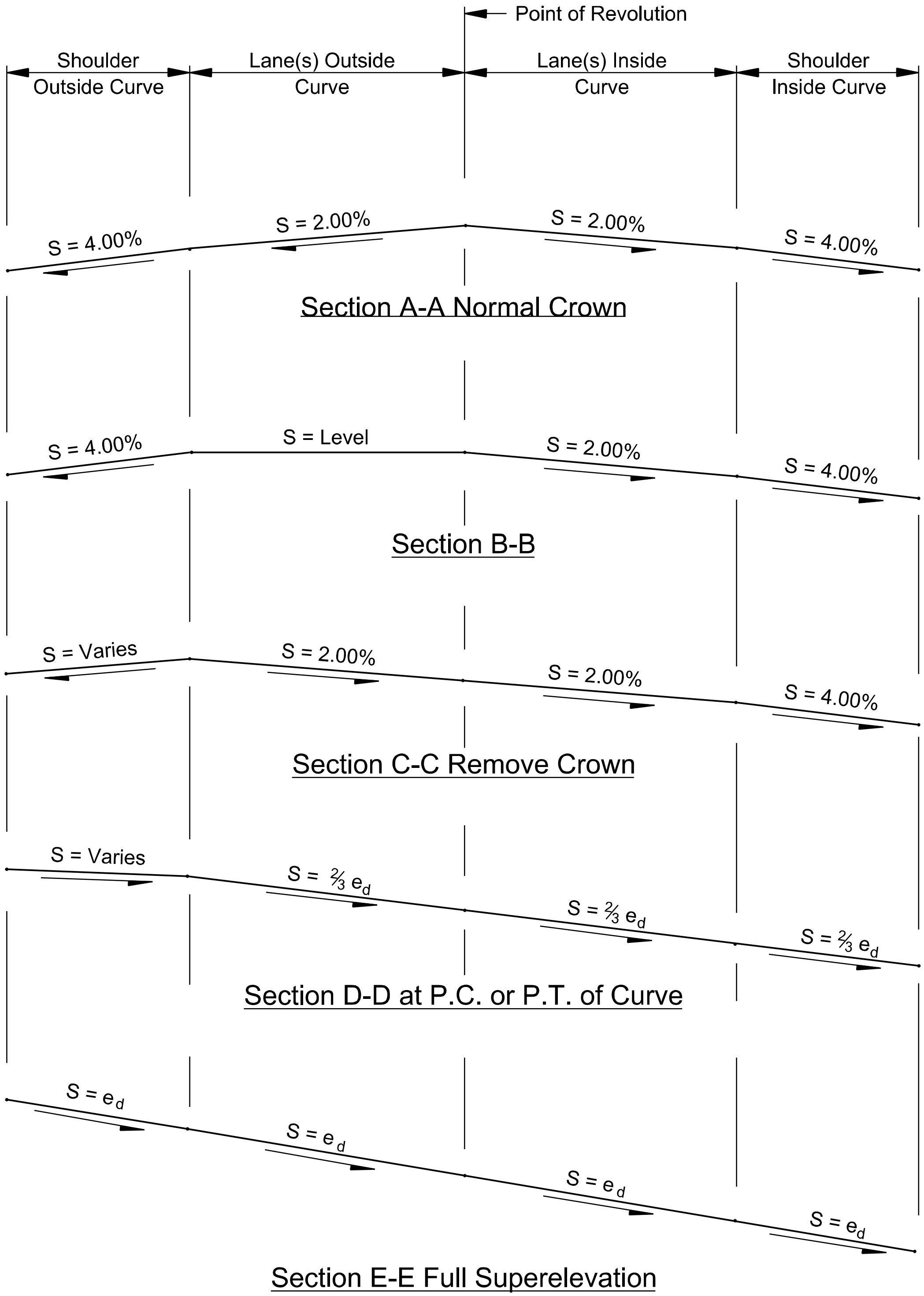
2. High Side. Retain the normal shoulder slope from the tangent section until where adjacent travel lane is level. From this point, transition the high-side shoulder until it matches the adjacent travel lane slope at the point where the design superelevation (e_d) is reached on the adjacent travel lane.

5.3.6 Compound Curves

As discussed in Section 5.2.2.3, compound curves should rarely be used on the mainline and, then, only two-centered curves should be used. When used, the development of superelevation requires special considerations. The designer should review the following criteria:

1. If the distance between the PC and PCC is less than or equal to 300 feet, use a uniform relative gradient throughout the superelevation transition length. Develop the superelevation so that, for the first curve, two-thirds of the design superelevation rate for this curve will be attained at the PC. Develop the superelevation so that, for the second curve, the full design superelevation rate (e_d) will be available at the PCC.

2. If the distance between the PC and PCC is more than 300 feet, it may be preferable to consider the two curves separately. Superelevation for the first curve is developed by the distribution method used for simple curves. This curve's superelevation rate will then be maintained until it is necessary to develop the remaining superelevation of the second curve consistent with the Department's superelevation development practices (e.g., relative gradient).



Note: See Section 5.3.5 for criteria on treatment of shoulders through superelevated curves.

SHOULDER TREATMENT THROUGH SUPERELEVATED CURVE
Figure 5.3-I

5.3.7 Reverse Curves

Reverse curves are two closely spaced simple curves with deflections in opposite directions. In some situations, because of the proximity of the curves, it is not possible to adhere to the usual superelevation development criteria for each curve and achieve a normal tangent section between the curves. For this case, the designer should use the following steps to adjust the superelevation development:

Step 1: Determine the point where the transitions should meet. The length of transition should favor the curve with the smaller (sharper) radius. Assume that, for the first iteration, the cross slope at this point is the normal crown (2.00 percent, typically).

Step 2: Determine the maximum relative gradient (Δ) from Figure 5.3-A.

Step 3: Apply the superelevation rate while maintaining Δ and work back from the point determined in Step 1.

Step 4: Examine the superelevation of the curves to ensure that:

- no more than 40 percent of the transitions occur in either curve, and
- the length of full superelevation on each curve is sufficient.

Step 5: If either of the criteria set forth in Step 4 are not met, recalculate the superelevation with the normal cross slope reduced to no less than 1 percent.

If the superelevation still cannot be developed properly, the designer should consider revising the alignment or possibly adjusting the design speed.

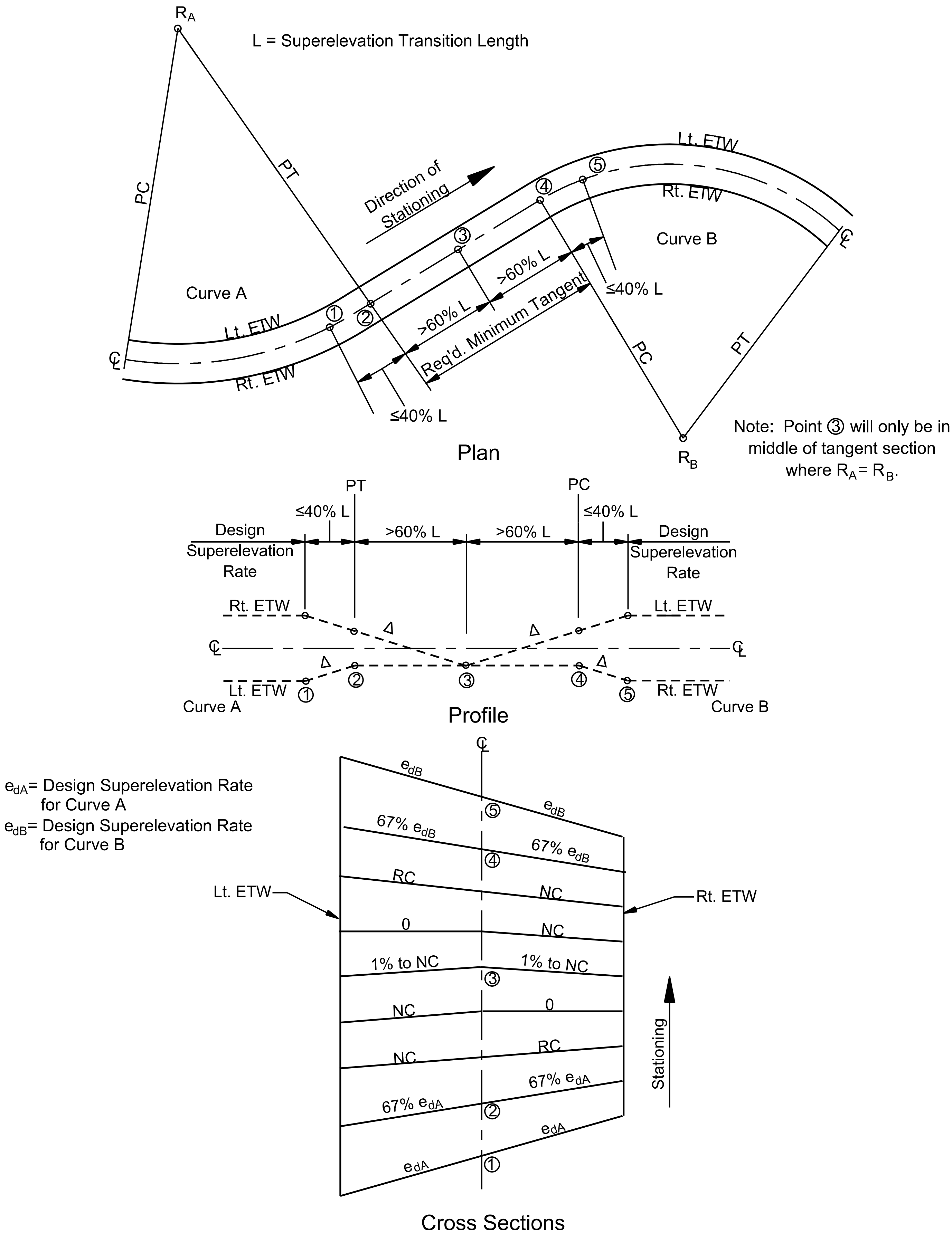
Figure 5.3-J illustrates superelevation development for reverse curves.

5.3.8 Bridges

From the perspective of the roadway user, a bridge is an integral part of the roadway system and, ideally, horizontal curves and their transitions will be located irrespective of their impact on bridges. However, practical factors in bridge design and bridge construction warrant consideration in the location of horizontal curves at bridges. The following presents, in order from the most desirable to the least desirable, the application of horizontal curves to bridges:

1. Considering both the complexity of design and construction difficulty, the most desirable treatment is to locate the bridge and its approach slabs on a tangent section (i.e., no portion of the curve or its superelevation development will be on the bridge or bridge approach slabs).

2. If a horizontal curve is located on a bridge, do not locate the superelevation transition on the bridge or its approach slabs. This will result in a uniform cross slope (i.e., the design superelevation rate) throughout the length of the bridge and bridge approach slabs.



SUPERELEVATION DEVELOPMENT FOR REVERSE CURVES
Figure 5.3-J

3. If the superelevation transition is located on the bridge or its approach slabs, coordinate with the structural designer. Typically, the designer should place on the roadway approach that portion of the superelevation development that transitions the roadway cross section from its normal crown to a point where the roadway slopes uniformly. This will avoid the need to warp the crown on the bridge or the bridge approach slabs.

5.3.9 Drainage

Two potential pavement surface drainage problems are of concern in the superelevation transition section. One problem relates to the potential lack of adequate longitudinal grade. This problem generally occurs where the longitudinal gradient of the point of revolution is equal to, but opposite in sign to, the relative longitudinal gradient for the superelevation transition length (e.g., 0.50 percent). It results in the edge of traveled way having negligible longitudinal grade that can lead to poor pavement surface drainage especially on curbed cross sections.

The second potential drainage problem relates to adequate lateral drainage due to negligible cross slope during pavement rotation. This problem occurs in the transition section where the cross slope of the outside lane varies from an adverse slope at the normal cross slope rate to a superelevated slope at the normal cross slope rate. This length of the transition section includes the tangent runout section and an equal length of the superelevation runoff section. Within this length, the pavement cross slope may not be sufficient to adequately drain the pavement laterally.

Two techniques can be used to alleviate these two potential drainage problems. One technique is to provide a minimum profile (finished) grade in the transition section. The second technique is to provide a minimum edge-of-traveled-way grade in the transition section. Both techniques can be incorporated in the design by use of the following criteria:

1. Maintain a minimum profile (finished) grade of 0.5 percent through the superelevation transition length.

2. Maintain a minimum edge-of-traveled-way grade of 0.2 percent (0.5 percent for curbed streets) through the superelevation transition length.

To illustrate the combined use of the two grade criteria, consider an uncurbed roadway curve having a relative gradient of +0.50 percent in the superelevation transition length entering the curve and –0.50 percent for the superelevation transition length exiting the curve. The first criterion would exclude finished grades between -0.50 and +0.50 percent. The second criterion would exclude finished grades in the range of –0.30 to –0.70 percent (entering) and those in the range of +0.30 to +0.70 percent (exiting). Given the overlap between the ranges for controls 1 and 2, the profile (finished) grade throughout the curve must be outside of the range of –0.70 to +0.70 percent to satisfy both criteria and provide adequate pavement surface drainage.

5.4 HORIZONTAL SIGHT DISTANCE

5.4.1 Sight Obstruction (Definition)

Sight obstructions on the inside of a horizontal curve are defined as obstructions that interfere with the line of sight on a continuous basis. In general, point obstacles (e.g., traffic signs, utility poles) are not considered sight obstructions on the inside of horizontal curves. The designer must examine each curve individually to determine whether it is necessary to acquire additional right of way to attain the required sight distance.

5.4.2 Length > Stopping Sight Distance

Where the length of curve (L) is greater than the stopping sight distance (SSD) used for design, calculate the needed clearance on the inside of the horizontal curve using the following equation:

$$HSO = R\left(1 - \cos\left[\frac{28.65\ SSD}{R}\right]\right) \qquad \text{(Equation 5.4-1)}$$

Where:

HSO = Horizontal sight line offset or distance from the center of the inside travel lane to the obstruction, feet

R = Radius of curve at the center of inside lane, feet

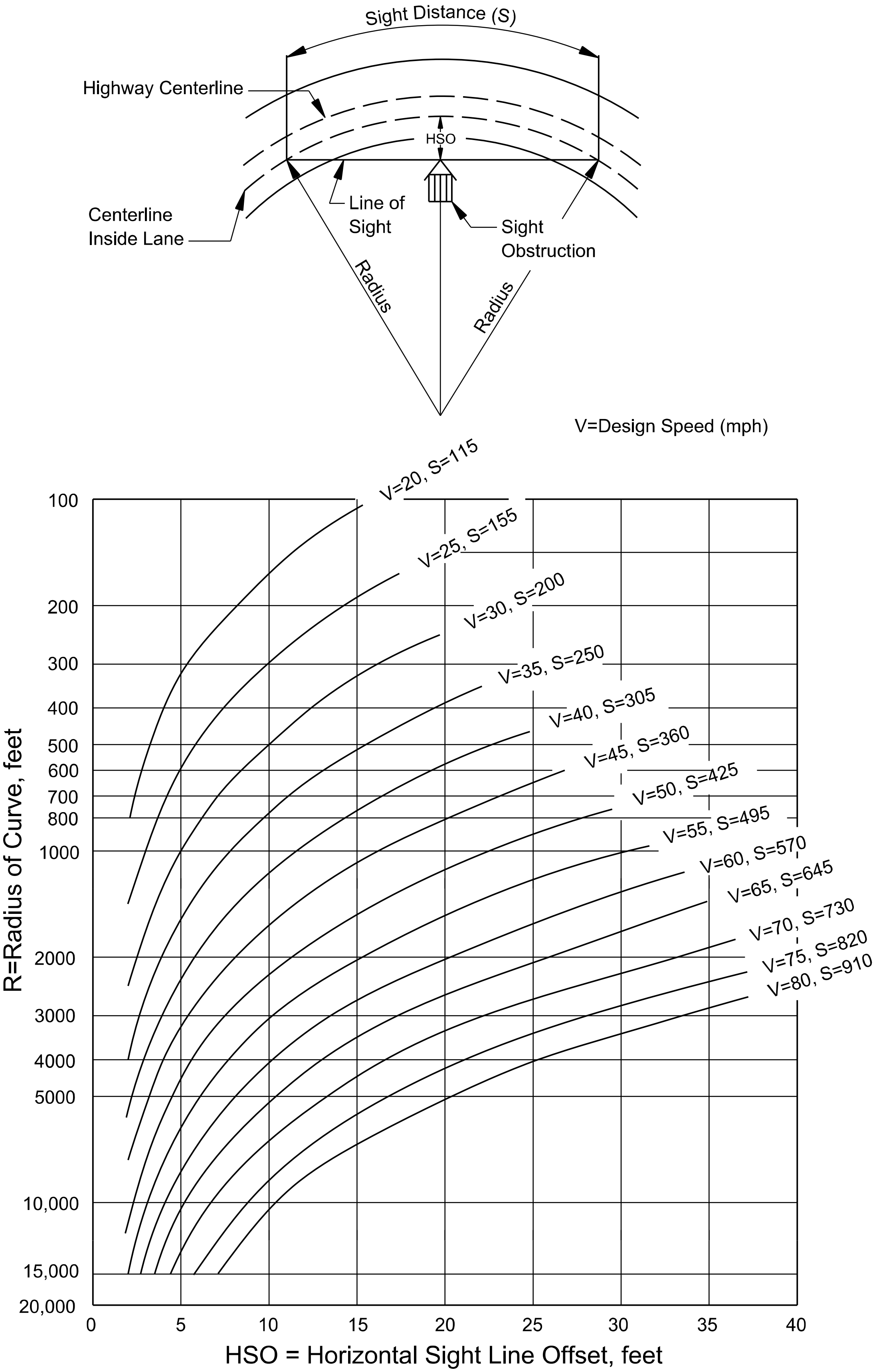
SSD = Stopping sight distance, feet

5.4.2.1 Stopping Sight Distance (SSD)

At a minimum, SSD will be available throughout the horizontal curve. The following discusses the application of SSD to sight distance at horizontal curves:

1. Level Grade. Figure 5.4-A provides the horizontal clearance criteria (i.e., middle ordinate) for various combinations of stopping sight distance (see Figure 4.1-A) and curve radii for level grades. For those selections of SSD that fall outside of the figures (i.e., HSO > 40 feet and/or R < 100 feet), the designer should use Equation 5.4-1 to calculate the needed clearance.

2. Grade Adjustment. Figure 4.1-C presents SSD values for passenger cars adjusted for 3 to 10 percent downgrades. If the downgrade on the facility is 3 percent or steeper, the designer should consider providing horizontal clearances adjusted for grade. Use the SSD values from Figure 4.1-C directly in Equation 5.4-1 to calculate the horizontal sight offset.



STOPPING SIGHT DISTANCE AT HORIZONTAL CURVES
(Level Grades)
Figure 5.4-A

5.4.2.2 Entering/Exiting Portions (Typical Application)

The HSO values from Figure 5.4-A apply between the PC and PT. In addition, some transition is needed on the entering and exiting portions of the curve. Example 5.4-1 (See Figure 5.4-B) illustrates the determination of clearance requirements for the entering and exiting portions of a curve. The designer should use the following steps:

Step 1: Locate the point that is on the outside edge of shoulder and a distance of SSD/2 before the PC.

Step 2: Locate the point that is a distance HSO measured laterally from the center of the inside travel lane at the PC.

Step 3: Connect the two points located in Steps 1 and 2. The area between this line and the roadway should be clear of all continuous obstructions.

Step 4: Use a symmetrical application of Steps 1 through 3 beyond the PT.

5.4.3 Length < Stopping Sight Distance

When the length of curve is less than the stopping sight distance used in design, the HSO value from the basic equation will never be reached. As an approximation, determine the horizontal clearance for these curves as follows:

Step 1: For the given R and SSD, calculate HSO assuming L > SSD.

Step 2: The maximum HSO′ value will be needed at a point of L/2 beyond the PC. HSO′ is calculated using the following proportion:

$$\frac{HSO'}{HSO} = \frac{1.2\,L}{SSD} \qquad \text{(Equation 5.4-2)}$$

$$HSO' = \frac{1.2(L)(HSO)}{SSD}$$

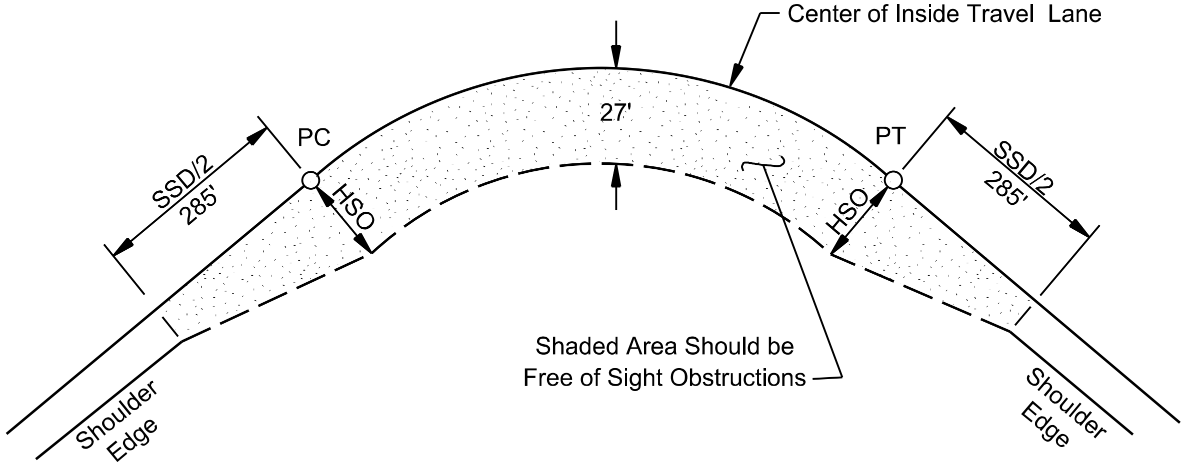
Where:

HSO′	=	Middle ordinate for a curve where L < SSD, feet
HSO	=	Middle ordinate for the curve based on Equation 5.4-1, feet
L	=	Length of the curve, feet
SSD	=	Stopping sight distance, feet

Step 3: Locate the point that is on the outside edge of shoulder and a distance of SSD/2 before the PC.

Step 4: Connect the two points located in Steps 2 and 3. The area between this line and the roadway should be clear of all continuous obstructions.

Step 5: Use a symmetrical application of Steps 2 through 4 on the exiting portion of curve.



Example 5.4-1

Given: Design Speed = 60 miles per hour
R = 1500 feet
Level Grade

Problem: Determine the horizontal clearance requirements for a horizontal curve on a two-lane highway using SSD.

Solution: Figure 4.1-A yields a SSD = 570 feet. Using Equation 5.4-1 for horizontal clearance (L > SSD):

$$HSO = R\left(1 - \cos\left[\frac{28.65\ SSD}{R}\right]\right)$$

$$HSO = 1500\left(1 - \cos\left[\frac{(28.65)(570)}{1500}\right]\right) = 27 \text{ feet}$$

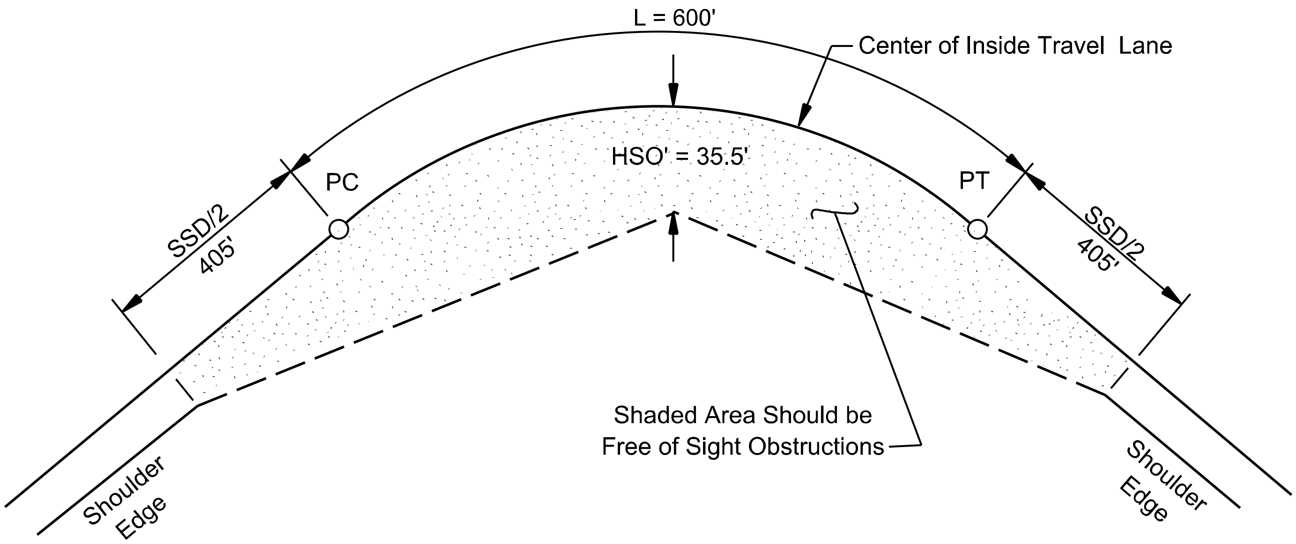
The above figure also illustrates the horizontal clearance requirements for the entering and exiting portion of the horizontal curve.

SIGHT CLEARANCE REQUIREMENTS FOR HORIZONTAL CURVES
(L > SSD)
Figure 5.4-B

Example 5.4-2 (See Figure 5.4-C) illustrates the determination of clearance requirements for the entering and exiting portions of a curve where L < SSD.

5.4.4 Application

For sight distance applications at horizontal curves, the height of eye is 3.5 feet and the height of object is 2 feet. Both the eye and object are assumed to be in the center of the inside travel lane. The line-of-sight intercept with the obstruction is at the midpoint of the sightline and 2.75 feet above the center of the inside lane.



<u>Example 5.4-2</u>

<u>Given</u>: Design Speed = 70 miles per hour
R = 2050 feet
L = 600 feet
Grade = 5.0 percent downgrade

<u>Problem</u>: Determine the clearance requirements for the horizontal curve on a two-lane highway.

<u>Solution</u>: Because the downgrade is greater than 3.0 percent, desirably adjust the curve for grade. Figure 4.1-C yields a SSD of 810 feet for 70 miles per hour and a 5.0 percent downgrade. Therefore, L < SSD (600 feet < 810 feet), and the horizontal clearance is calculated from Equation 5.4-2 as follows:

$$\text{HSO (L} > \text{SSD)} = 2050\left[1 - \cos\frac{(28.65)(810)}{2050}\right] = 39.88 \text{ feet}$$

$$\text{HSO}' \ (\text{L} < \text{SSD}) = \frac{1.2(600)(39.88)}{810}$$

$$\text{HSO}' = 35.5 \text{ feet}$$

Therefore, a minimum clearance of 35.5 feet should be provided at a distance of L/2 = 300 feet beyond the PC. The obstruction-free triangle around the horizontal curve would be defined by HSO′ (35.5 feet) at L/2 and by points at the shoulder edge at SSD/2 = 405 feet before the PC and beyond the PT.

SIGHT CLEARANCE REQUIREMENTS FOR HORIZONTAL CURVES
(L < SSD)
Figure 5.4-C

5.5 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

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Chapter 6

VERTICAL ALIGNMENT

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

SPACER PAGE

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Chapter 6
VERTICAL ALIGNMENT

The vertical alignment contributes significantly to a highway's safety, aesthetics, operations and costs. Long, gentle vertical curves provide greater sight distances and a more pleasing appearance for the driver. Chapters 14 through 18 provide numerical criteria for various vertical alignment elements based on highway functional classification, project scope of work and urban/rural environment. This chapter provides guidance on developing a profile grade line, maximum and minimum grades, critical lengths of grade, truck-climbing lanes, vertical curvature and vertical clearances.

6.1 DEFINITIONS

1. Broken-Back Curves. A grade line with two vertical curves in the same direction separated by a short section of tangent grade.

2. Critical Length of Grade. The maximum length of a specific upgrade on which a loaded truck can operate without an unreasonable reduction in speed.

3. Grade Slopes. The slope expressed as a percent between two adjacent VPIs. The numerical value for the grade is the vertical rise or fall in feet for each foot of horizontal distance. The numerical value is multiplied by 100 and is expressed as a percent. Upgrades in the direction of stationing are identified as positive (+). Downgrades are identified as negative (-).

4. K-Values. The horizontal distance needed to produce a 1 percent change in gradient.

5. Level Terrain. Level terrain generally is considered to be flat and has minimal impact on vehicular performance. Highway sight distances in level terrain generally can be made longer without major construction expense.

6. Momentum Grade. Sites where an upgrade is preceded by a downgrade. These locations allow a truck to increase its speed on the downgrade before ascending the upgrade.

7. Mountainous Terrain. Locations where longitudinal and transverse changes in elevation are abrupt, and benching and side hill excavation are usually required to provide the desirable horizontal and vertical alignment. Mountainous terrain aggravates the performance of trucks relative to passenger cars and results in some trucks operating at crawl speeds.

8. Performance Curves. A set of curves that illustrate the effect grades will have on the design vehicle's acceleration and/or deceleration.

9. Profile Grade Line. A series of tangent lines connected by vertical curves. Typically, the grade line is located along the roadway centerline of undivided multilane facilities and two-lane, two-way highways.

10. Rolling Terrain. Locations where the natural slopes consistently rise above and fall below the roadway grade line and, occasionally, steep grades present some restriction to the desirable horizontal and vertical alignment. In general, rolling terrain generates steeper grades causing trucks to reduce speeds below those of passenger cars.

11. VPC (Vertical Point of Curvature). The point at which a tangent grade ends and the vertical curve begins.

12. VPI (Vertical Point of Intersection). The point where the extension of two tangent grades intersect.

13. VPT (Vertical Point of Tangency). The point at which the vertical curve ends and the tangent grade begins.

6.2 DESIGN PRINCIPLES AND PROCEDURES

6.2.1 General Controls for Vertical Alignment

The design of vertical alignment involves, to a large extent, complying with specific limiting criteria. These criteria include maximum and minimum grades, sight distance at vertical curves and vertical clearances. In addition, the designer should adhere to certain general design principles and controls that will determine the overall safety and operation of the facility and will enhance the aesthetic appearance of the highway. These design principles for vertical alignment include:

1. Consistency. Use a smooth grade line with gradual changes, consistent with the type of highway and terrain character.

2. Coordination with Natural/Man-Made Features. Coordinate the vertical alignment with the natural topography, available right of way, utilities, roadside development and natural/man-made drainage patterns. This is especially important in mountainous terrain.

3. Roller Coaster Profile. Avoid a roller-coaster or hidden-dip type of profile, especially where the horizontal alignment is relatively straight. Hidden dips may create difficulties for drivers who wish to pass, because the passing driver may be deceived if the view of the road or street beyond the dip is free of opposing vehicles. To avoid this type of profile, incorporate horizontal curvature and/or flatter grades that may require greater excavations and higher embankments into the design.

4. Broken-Back Curvature. Avoid broken-back grade lines (two crest or sag vertical curves separated by a short tangent). This alignment is particularly noticeable on divided highways with open-ditch median sections. One long vertical curve is more desirable.

5. Long Grades. On a long ascending grade, it is preferable to place the steepest grade at the bottom and flatten the grade near the top. It is also preferable to break the sustained grade with short intervals of flatter grades. Evaluate substantial lengths of grades for their effect on traffic operations (e.g., trucks).

6. Sags. Avoid sag vertical curves in cuts unless adequate drainage can be provided. In addition, avoid placing the low point of a sag vertical curve on bridges and approaches. See the *SCDOT Bridge Design Manual* for additional guidance.

7. Intersections. Where intersections occur on roadway sections with moderate to steep grades, it is desirable to reduce the grade through the intersection. This will help facilitate vehicular braking and turning movements. See Section 9.2.7 for specific information on vertical alignment through intersections.

8. Environmental Impacts. Vertical alignment should be properly coordinated with environmental impacts. However, the safety of motorists using the highway should not be compromised.

6.2.2 Coordination of Horizontal and Vertical Alignment

Do not design the horizontal and vertical alignments independently. Instead, they should complement each other. This is especially true for new construction projects. A thorough study of the alignment is always warranted.

Horizontal and vertical alignments are among the most important design elements for a highway. Excellence in their design and coordination increases the highway's utility and safety, encourages uniform speeds and can greatly improve the highway's appearance. This usually can be accomplished with little additional costs. The designer should coordinate the layout of the horizontal and vertical alignment as early as practical in the design process.

In addition, consider the following when coordinating horizontal and vertical alignment on rural and suburban highways:

1. Balance. Horizontal curvature and grades should be in proper balance. Sharp curvature with flat grades or flat curvature with maximum grades does not achieve this desired balance. A compromise between the two extremes produces the best design relative to safety, capacity, ease and uniformity of operations and aesthetics.

2. Coordination. Superimposing the vertical curvature upon horizontal curvature (i.e., vertical and horizontal PIs at approximately the same stations) generally results in a more pleasing appearance and reduces the number of sight distance restrictions. Successive changes in profile not in combination with horizontal curvature may result in a series of humps visible to the driver for some distance, which may produce an unattractive design. However, in some circumstances, superimposing the horizontal and vertical alignment must be tempered somewhat by Comments 3 and 4 below.

3. Crest Vertical Curves. Introducing sharp horizontal curvature at or near the top of pronounced crest vertical curves is undesirable because the driver cannot perceive the horizontal change in alignment, especially at night when headlight beams project straight ahead into space. To improve this condition, use horizontal curvature that leads the vertical curvature or use design values that well exceed the minimums.

4. Sag Vertical Curves. Introducing sharp horizontal curves at or near the low point of pronounced sag vertical curves or at the bottom of steep grades is undesirable because visibility of the road ahead is reduced. To improve this condition use flat horizontal curvature to avoid an undesirable, distorted appearance. At the bottom of long grades, vehicular speeds often are higher, particularly for trucks, and erratic operations may occur, especially at night and during icy conditions.

5. Passing Sight Distance. In some cases, the need for frequent passing opportunities and a higher percentage of passing sight distance may supersede the desirability of combining horizontal and vertical alignment. In these cases, it may be necessary to provide long tangent sections to secure sufficient passing sight distance; see Section 4.2.

6. Superelevation. Avoid flat areas that hinder proper drainage, especially where superelevation transitions coincide with the top of crests, bottom of sags or relatively flat tangent grade sections. In addition, when vertical and horizontal curves are

superimposed, the superelevation may cause a distortion in the outer pavement edges. For curb and gutter sections, plot and review the profile along the top of curb and remove any irregularities with a smooth vertical curve.

7. Intersections. At intersections, horizontal and vertical alignment should be as flat as practical to provide a design that produces sufficient sight distance and gradients for vehicles to slow, stop or turn; see Chapter 9 "Intersections."

8. Divided Highways. On divided facilities with wide medians, it may be advantageous to provide independent alignments for the two one-way roadways. Where traffic volumes justify a divided facility and rolling or rugged terrain exists, a superior design can result from the use of independent alignments and profiles.

9. Residential Areas. For highways near subdivisions, design the alignment and profile to minimize nuisance factors to neighborhoods. For freeways, a depressed facility can make the highway less visible and reduce the noise to adjacent residents. In addition, for all highway types, minor adjustments to the horizontal alignment may increase the buffer zone between the highway and residential areas.

10. Aesthetics. Design the alignment to enhance attractive scenic views of rivers, rock formations, parks, golf courses, etc. The highway should head into rather than away from those views that are considered to be aesthetically pleasing. The highway should fall towards those features of interest at a low elevation and rise toward those features that are best seen from below or in silhouette against the sky.

6.2.3 Design of Profile Grade Lines

6.2.3.1 General

The profile grade line of a highway typically has the greatest impact on a facility's cost, aesthetics, safety and operation. The profile is a series of tangent lines connected by parabolic vertical curves.

The designer must carefully evaluate many factors when establishing the profile grade line of a highway. These include:

- maximum and minimum gradients;
- sight distance criteria;
- earthwork balance;
- bridges and drainage structures;
- high-water levels (flood frequency);
- drainage considerations;
- water table elevations;
- intersections and interchanges;
- railroad/highway crossings;
- types of soil;
- adjacent land use and values;
- highway safety;
- coordination with other geometric features (e.g., the cross section);

- topography/terrain;
- truck performance;
- available right of way and associated costs;
- type and location of utilities;
- urban/rural location;
- aesthetics/landscaping;
- construction costs;
- environmental impacts;
- driver expectations;
- airport flight paths (e.g., grades and lighting); and
- pedestrians and accessibility.

The following sections discuss the establishment of the profile grade line in more detail.

6.2.3.2 Profile Grade Line Locations

The location of the profile grade line on the roadway cross section varies according to the highway and median type. The profile grade line locations are shown in the typical cross section figures provided in Chapters 14 through 17. The profile grade line should generally coincide with the point of revolution for superelevation. The recommended profile grade line for various typical sections are as follows:

1. Flush Medians. The profile grade line should coincide with the highway centerline.

2. Depressed Medians. The profile grade lines should be located at the point of grade on each of the traveled ways. The grade on each of the traveled ways may be independent of each other when the median width varies or on freeways where the terrain is conducive to independent roadway designs. Separate horizontal alignments also may need to be developed.

3. Ramps/Freeway to Freeway Connections. The profile grade line is normally established on the survey line, but may be positioned at either edge of the ramp traveled way or the centerline on multilane ramps.

6.2.3.3 Factors Affecting the Design of Profile Grade Lines

Consider the following factors when developing a profile grade line on a project:

1. Urban Streets. Long vertical curves on urban streets are generally impractical. The designer will typically need to lay out the profile grade line to meet existing conditions. Therefore, minimum vertical curve lengths generally are provided on urban streets; see Sections 6.5.1 and 6.5.2 (i.e., L = 3V). Where practical, locate VPIs at or near the centerlines of cross streets. At signalized and stop-controlled intersections, some flattening of the approaches also may be required for better traffic operations.

2. Spline Grades. Spline grades are series of grade breaks in the profile grade line between short lengths of tangents and no vertical curves. Spline grades can be helpful in laying grades in urban areas where it is necessary to meet numerous elevation

restrictions in relatively short distances, in gore areas to ensure proper drainage, to meet the existing roadway at the project beginning and ending termini, and for determining the outside edge of right-turn lanes where the main lane is on a horizontal curve.

For spline grades on the profile grade line, grade breaks should not be closer than 100 feet and the total of all grade breaks within 200 feet should not exceed 1 percent on urban streets with design speed 35 miles per hour or less.

3. Grade Differential. Avoid appreciable grade differentials between roadways on divided facilities, for either interim or ultimate designs, in the vicinity of intersections. Confusion and/or wrong-way movements could result for traffic entering from the crossroad if the pavement surface of the roadway on the opposite side is obscured from view.

4. Soils. The type of material encountered often influences the profile grade line at certain locations. For example, if rock is encountered, it may be more economical to raise the grade line and reduce the rock excavation. Soils that are unsatisfactory for embankment or cause a stability problem in cut areas may also be determining factors in establishing the profile grade line. The designer should coordinate the development of the profile grade line with the geotechnical designer.

5. Drainage. Proper placement of the pavement structure above the surrounding topography can significantly enhance the life and serviceability of the roadway. Consequently, the profile grade line should be compatible with the roadway drainage design. The designer should consider the following:

   a. Freeboard. To protect the roadway pavement, it is recommended that the road subgrade be 1 foot above the design high-water level.

   b. Culverts. The roadway elevation should meet the Department criteria for minimum cover at culverts and minimum freeboard above the headwater level at culverts. See the *SCDOT Requirements for Hydraulic Design Studies* for more information on the hydraulic and structural design of culverts.

   c. Coordination with Geometrics. The profile grade line must reflect compatibility between drainage design and roadway geometrics. Items to consider include the design of sag and crest vertical curves, spacing of inlets on curbed facilities, impacts on adjacent properties, superelevated curves, intersection design elements and interchange design elements.

   d. Curb and Gutter. Curb and gutter may complicate the layout of the profile grade line. Take special care to avoid flat spots where water may pond, especially through intersection radius returns. Section 6.3 provides the minimum gradients for curbed streets. In very flat areas, the profile grade line may be rolled up and down at 0.3 to 0.5 percent to provide the necessary drainage. At intersections, the surface drainage preferably should be intercepted upstream of an intersection. Ensure adequate drainage is provided behind curbs.

   e. Existing Drainage. It is beneficial to coordinate the low point of vertical curves with existing terrain low points.

6. Erosion Control. To minimize erosion, consider the following relative to the profile grade line:

- Minimize the number of deep cuts and high-fill sections.
- Conform the highway to the contour and drainage patterns of the area.
- Use natural land barriers and contours to channelize runoff and confine erosion and sedimentation.
- Minimize the amount of disturbance.
- Preserve and use existing vegetation.
- Reduce the slope length by using slope interruption devices as discussed in the SCDOT Supplemental Technical Specifications.
- Ensure that erosion is confined to the right of way and does not deposit sediment or erode adjacent lands.
- Avoid locations having high erosion potential (e.g., loose soils).
- Avoid cut or fill sections in seepage areas.

7. Earthwork Balance. Where practical and consistent with other project objectives, design the profile grade line to provide a balance of earthwork. However, this should not be achieved at the expense of smooth grade lines, aesthetics, sight distance requirements at vertical curves, or when there are excessive land acquisition costs. Ultimately, a project-by-project assessment will determine whether a project will be borrow, waste or balanced.

8. Bridges. Carefully coordinate the design of the profile grade line with any bridges within the project limits. The following will apply:

   a. Vertical Clearances. The criteria in this chapter and Chapters 14 through 18 must be met. When laying the preliminary grade line, an important element in determining the available vertical clearance is the assumed structure depth. This will be based on the structure type, span lengths and depth/span ratio. The designer should coordinate with the bridge designer to determine the roadway and bridge grade lines.

   b. Bridges Over Waterways. Where a proposed facility will cross a body of water, the bridge elevation must be consistent with the necessary waterway opening to meet the Department's hydraulic requirements. Coordinate with the hydraulic designer and bridge designer to determine the appropriate bridge elevation.

   c. Railroad Bridges. Any proposed highway over a railroad must meet the applicable criteria (e.g., vertical clearances, structure type and depth). The designer should contact the Utilities Office for more information.

d. Highway under Bridge. When practical, the low point of a roadway sag vertical curve should not be within the shadow of the bridge. This will help minimize ice accumulations and reduce the ponding of water beneath the bridge. To achieve these objectives, the low point of roadway sag should be approximately 100 feet or more from the side of the bridge.

e. High Embankments. Consider the impact that high embankments will have on bridges and culverts. High embankments will increase the span length thereby increasing structure costs, and also increase the length and type of culvert to carry the overburden.

f. Sag Vertical Curves. If practical, do no place any portion of a bridge within a sag vertical curve. If the bridge is in a sag vertical curve, avoid placing the low point on the bridge and approach slabs. Place the low point a minimum of 10 feet from the end of the approach slab or, if approach slabs are not used, a minimum of 10 feet from the end of the bridge. See the *SCDOT Bridge Design Manual* for additional guidance.

g. Minimum Grade. The minimum grade should be limited to 0.3 percent across the bridge.

9. Ties with Existing Highways. A smooth transition is needed between the proposed profile grade line of the project and the existing grade line of an adjoining highway section. Consider existing grade lines for a sufficient distance beyond the beginning and end of a project to ensure adequate sight distances. Connections should be made that are compatible with the design speed of the new project and that can be used if the adjoining road section is reconstructed.

10. Underground Utilities. On existing streets, ensure that any change in the profile grade line will still provide the minimum coverage for utilities. For additional guidance on minimum utility clearances, see the SCDOT publication *A Policy for Accommodating Utilities on Highway Rights of Way*.

11. Right of Way. Give careful consideration when substantially lowering or raising the profile grade line. This will often result in more right of way impacts (e.g., steeper driveways, removing parking, reducing front lawns, adding retaining walls). Where roadside development is extensive, the cross section design of a curb and gutter street is important.

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6.3 GRADES

6.3.1 Maximum Grades

Chapters 14 through 18 present the Department's criteria for maximum grades based on functional classification, urban/rural location, type of terrain, design speed and project scope of work. Wherever practical, use grades flatter than the maximum.

6.3.2 Minimum Grades

The following provides the Department's criteria for minimum grades:

1. Roadways without Curbs. It is desirable to provide a minimum longitudinal gradient of approximately 0.5 percent. This allows for the possibility of alterations to the original pavement. Longitudinal gradients of 0 percent may be acceptable on some pavements that have adequate cross slopes, and in locations where superelevation does not occur. In these cases, check the flow lines of the outside ditches to ensure adequate drainage.

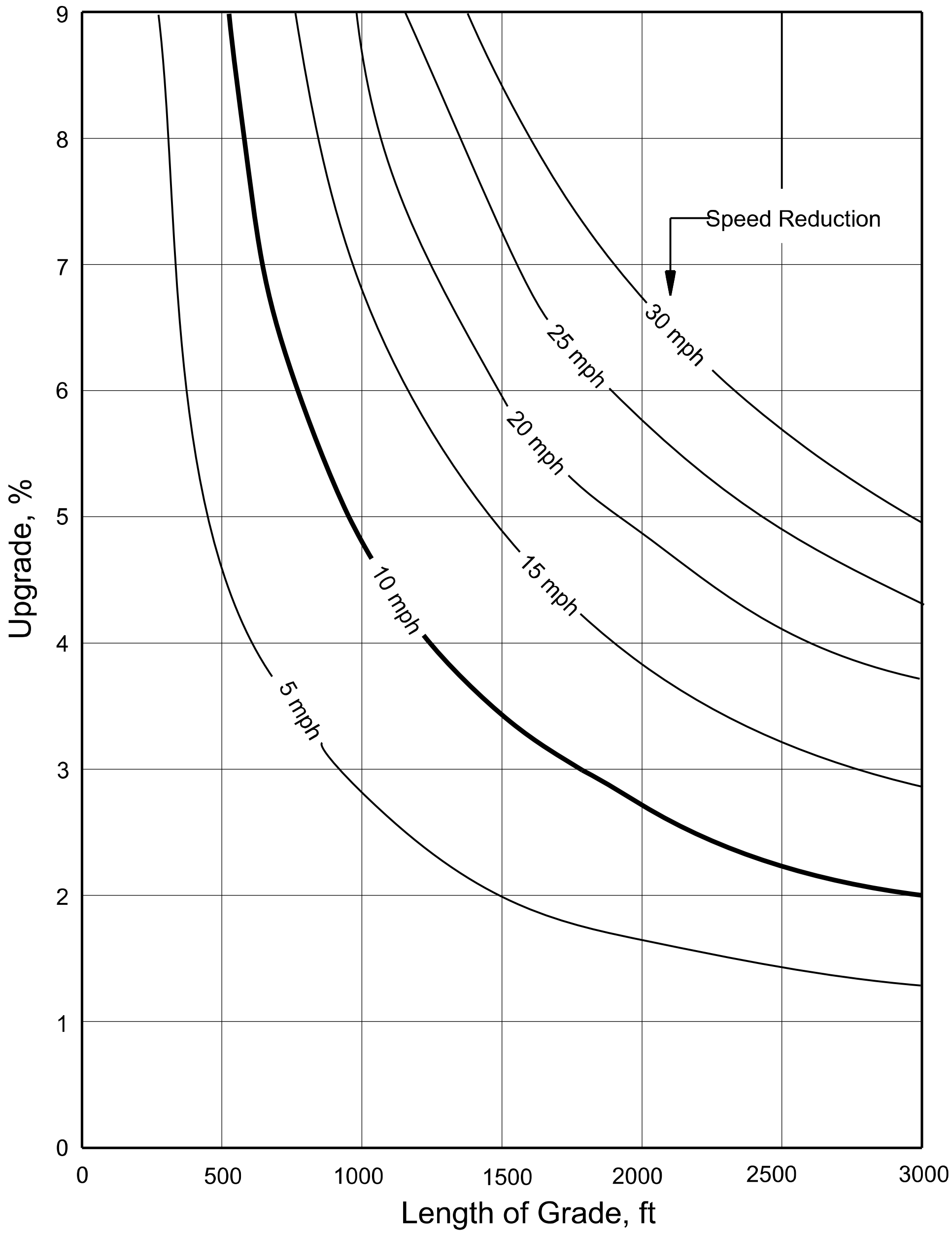
2. Roadways with Curbs/Valley Gutters. The median edge or centerline profile of streets with curb and gutter desirably should have a minimum longitudinal gradient of 0.5 percent. Where the adjacent development or flatter terrain precludes the use of a profile with a 0.5 percent grade, provide a minimum longitudinal gradient of at least 0.3 percent. Because surface drainage is retained within the roadway, the longitudinal gradient must be steeper on curb sections to avoid ponding of water on the roadway surface. Where additional catch basins are not feasible, trench drains may be installed in the gutters to enhance the drainage of the roadway. For additional information on trench drains, see Section 3.10.3.8.

6.3.3 Critical Length of Grade

The critical length of grade is the maximum length of a specific upgrade on which a truck can operate without an unreasonable reduction in speed. The highway gradient in combination with the length of the grade will determine the truck speed reduction on upgrades. For additional guidance, see the TRB *Highway Capacity Manual* and the AASHTO *A Policy on Geometric Design of Highways and Streets*.

The following will apply to the critical length of grade:

1. Design Vehicle. Use the 200 pound per horsepower truck for all truck routes in South Carolina. Figure 6.3-A presents the critical length of grade for a 200 pound per horsepower truck. In some instances, the recreational vehicle may be used as the design vehicle. Figure 6.3-B presents the critical length of grade for a recreational vehicle.



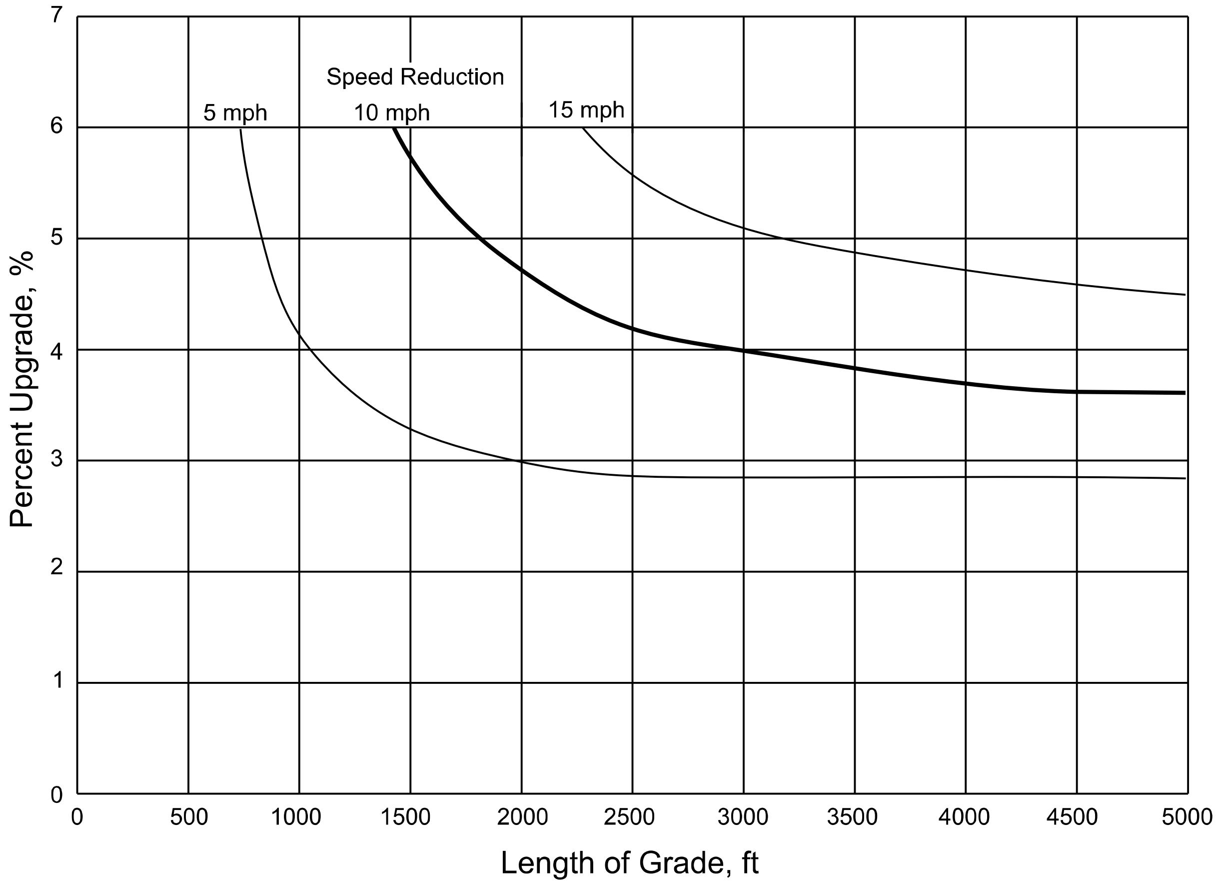
Notes:

1. *Typically, the 10 mile-per-hour curve will be used.*

2. *See examples in Section 6.3.3 for use of figure.*

3. *Figure is based on a truck with initial speed of 70 miles per hour. However, it may be used for any design or posted speed.*

CRITICAL LENGTH OF GRADE
(200 lb/hp Truck)
Figure 6.3-A



Notes:

1. *Typically, the 10 mile-per-hour curve will be used.*

2. *Figure is based on a recreation vehicle with initial speed of 55 miles per hour. However, it may be used for any design or posted speed.*

CRITICAL LENGTH OF GRADE
(Recreation Vehicle)
Figure 6.3-B

2. Criteria. Figures 6.3-A and 6.3-B provide the critical lengths of grade for a given percent grade and acceptable speed reduction. Although Figure 6.3-A is based on an initial truck speed of 70 miles per hour and Figure 6.3-B on an initial recreational vehicle speed of 55 miles per hour, they apply to any design or posted speed. For design purposes, use the 10 mile-per-hour speed reduction curve in the figures to determine if the critical length of grade is exceeded.

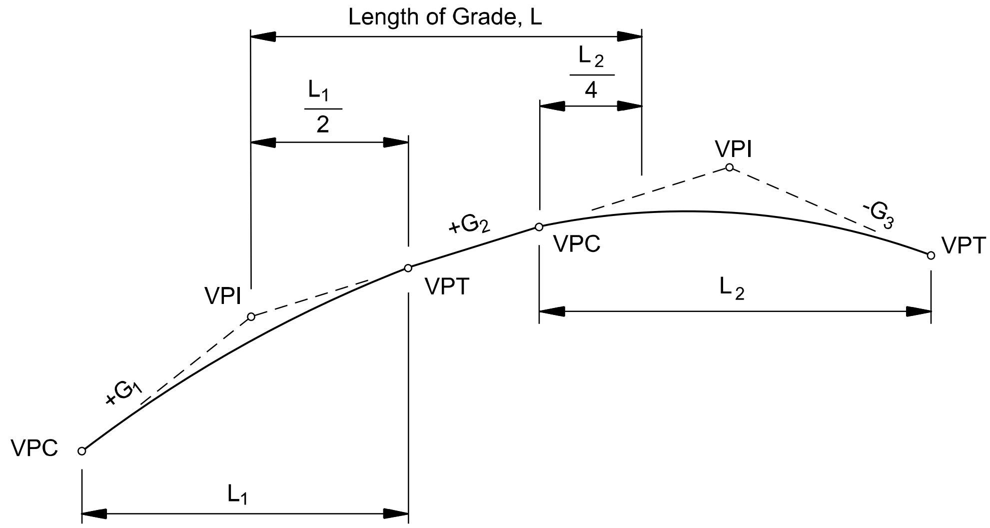
3. Momentum Grades. Where an upgrade is preceded by a downgrade, trucks will often increase their speed to ascend the upgrade. A speed increase of 5 miles per hour on moderate downgrades (3 percent to 5 percent) and 10 miles per hour on steeper downgrades (6 percent to 8 percent) of sufficient length are reasonable adjustments to the initial speed. This assumption allows the use of a higher speed reduction curve in Figure 6.3-A, which may indicate that a climbing lane may not be required. The designer should also consider that these speed increases may not always be attainable. If traffic volumes are sufficiently high, a truck may be behind another vehicle when descending the momentum grade thereby restricting the increase in speed. Therefore, only consider these increases in speed if the highway has a level of service B or better.

4. Measurement. Vertical curves are part of the length of grade. Figure 6.3-C illustrates how to measure the length of grade to determine the critical length of grade using Figure 6.3-A.

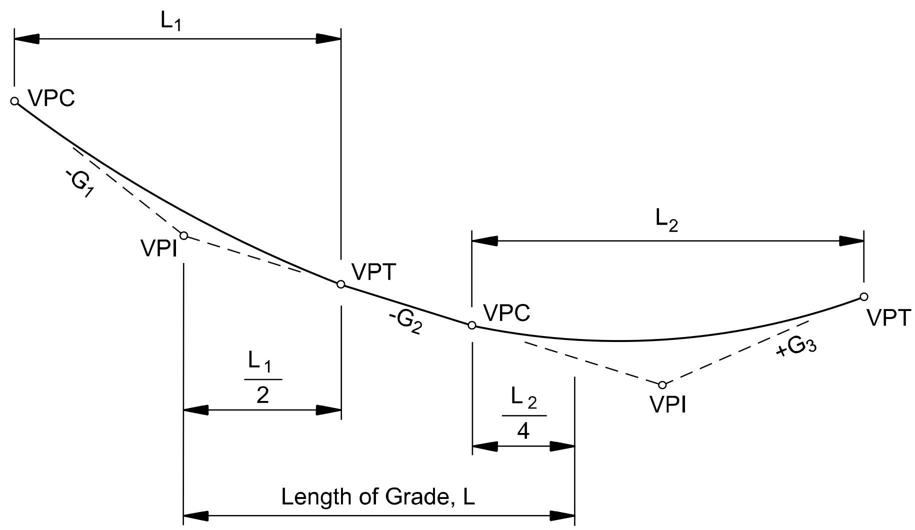
5. Application. If the critical length of grade is exceeded, either flatten the grade, if practical, or evaluate the need for a truck-climbing lane; see Section 6.4.

6. Highway Types. The critical-length-of-grade criteria apply equally to two-lane or multilane highways and apply equally to urban and rural facilities.

7. Example Problems. Examples 6.3-1 and 6.3-2 illustrate the use of Figure 6.3-A to determine the critical length of grade. Example 6.3-3 illustrates the use of Figures 6.3-A and 6.3-C.



Crest Vertical Curve



Sag Vertical Curve

Notes:

1. *For vertical curves where the two tangent grades are in the same direction (both upgrades or both downgrades), 50 percent of the curve length will be part of the length of grade.*

2. *For vertical curves where the two tangent grades are in opposite directions (one grade up and one grade down), 25 percent of the curve length will be part of the length of grade.*

3. *The above diagram is included for illustrative purposes only. Broken back vertical curves are to be avoided where practical.*

MEASUREMENT FOR LENGTH OF GRADE
Figure 6.3-C

* * * * * * * * * *

Example 6.3-1

Given: Level Approach
$G = +4$ percent
$L = 1500$ feet (length of grade)
Rural Principal Arterial

Problem: Determine if the critical length of grade is exceeded.

Solution: Figure 6.3-A yields a critical length of grade of 1250 feet for a 10 mile-per-hour speed reduction. The length of grade (L) exceeds this value. Therefore, flatten the grade, if practical, or evaluate the need for a truck-climbing lane.

Example 6.3-2

Given: Level Approach
$G_1 = +4.5$ percent
$L_1 = 500$ feet
$G_2 = +2$ percent
$L_2 = 700$ feet
Rural Collector with a significant number of heavy trucks

Problem: Determine if the critical length of grade is exceeded for the combination of grades G_1 and G_2.

Solution: From Figure 6.3-A, G_1 yields a truck speed reduction of approximately 5 miles per hour. G_2 yields a speed reduction less than 5 miles per hour. This results in a total less than the maximum 10 mile-per-hour speed reduction. Therefore, the critical length of grade is not exceeded.

Example 6.3-3

Given: Figure 6.3-D illustrates the vertical alignment on a low-volume, two-lane rural collector highway with sufficient number of large trucks to govern the design.

Problem: Determine if the critical length of grade is exceeded for G_2 or for the combination upgrade G_3 and G_4.

Solution: Use the following steps:

Step 1: Determine the length of grade using the criteria in Figure 6.3-C. For this example, these are calculated as follows:

$$L_2 = \frac{1000}{4} + 600 + \frac{800}{4} = 1050 \text{ feet}$$

$$L_3 = \frac{800}{4} + 700 + \frac{400}{2} = 1100 \text{ feet}$$

$$L_4 = \frac{400}{2} + 300 + \frac{600}{4} = 650 \text{ feet}$$

Step 2: Determine the critical length of grade in both directions. For trucks, use Figure 6.3-A to determine the critical length of grade.

a. For trucks traveling left to right, enter into Figure 6.3-A the value for G_3 (3.5 percent) and L_3 = 1100 feet. The speed reduction is approximately 7.5 miles per hour. For G_4 (2 percent) and L_4 = 650 feet, the speed reduction is approximately 3.5 miles per hour. The total speed reduction on the combination upgrade G_3 and G_4 is approximately 11 miles per hour. This exceeds the maximum 10 mile-per-hour speed reduction. However, on low-volume roads, one can assume a 5 mile-per-hour increase in truck speed for the 3 percent momentum grade (G_2), which precedes G_3. Therefore, a speed reduction may be as high as 15 miles per hour before concluding that the combination grade exceeds the critical length of grade. Assuming the benefits of the momentum grade, this leads to the conclusion that the critical length of grade is not exceeded.

b. For trucks traveling in the opposite direction, on Figure 6.3-A, enter in the value for G_2 (3 percent) and determine the critical length of grade for the 10 mile-per-hour speed reduction (i.e., 1700 feet). Because L_2 is less than 1700 feet (i.e., 1050 feet), the critical length of grade for this direction is not exceeded.

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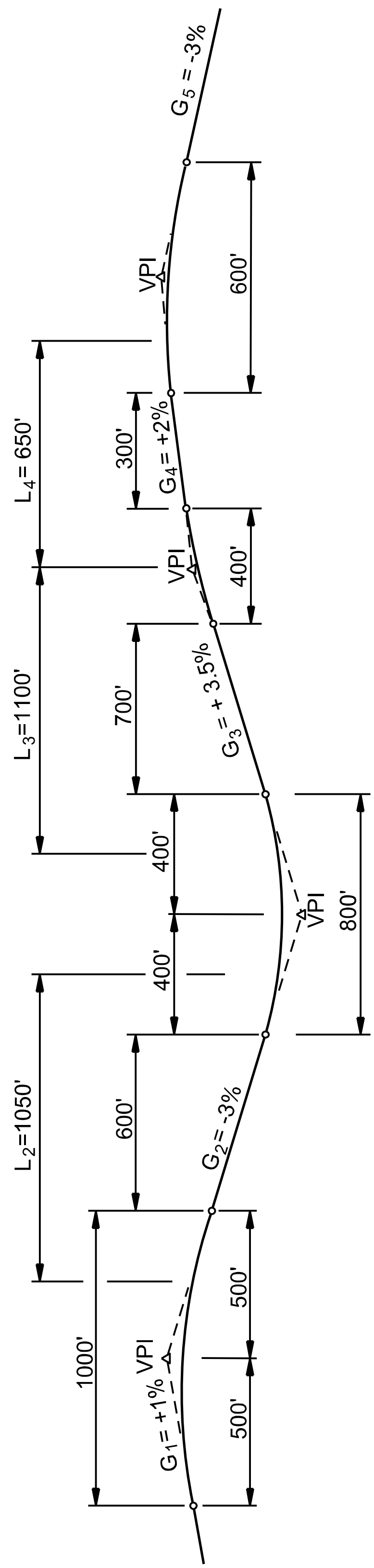


CRITICAL LENGTH OF GRADE CALCULATIONS
(Example 6.3-3)
Figure 6.3-D

6.4 TRUCK-CLIMBING LANES

6.4.1 Guidelines

Climbing lanes offer a comparatively inexpensive means of overcoming reduction in capacity and providing improved operation where congestion on grades is caused by slow trucks in combination with high traffic volume. On some two-lane highways, climbing lanes could defer reconstruction for many years or indefinitely. A truck-climbing lane may be necessary to allow a specific upgrade to operate at an acceptable level of service.

6.4.1.1 Two-Lane Highways

On a two-lane, two-way highway, consider a truck-climbing lane if the following conditions are satisfied:

- the up-grade traffic volume exceeds 200 vehicles per hour during the design hour; <u>and</u>
- the up-grade heavy-vehicle volume (i.e., trucks, buses and recreational vehicles) exceeds 20 vehicles per hour during the design hour; <u>and</u>
- the construction costs and the construction impacts (e.g., environmental, right of way) are considered reasonable; <u>and</u>
- one of the following conditions exists:
  - \+ the critical length of grade is exceeded for the 10 mile-per-hour speed reduction curve (see Figure 6.3-A or Figure 6.3-B); <u>or</u>
  - \+ the level of service (LOS) on the upgrade is E or F; <u>or</u>
  - \+ there is a reduction of two or more LOS when moving from the approach segment to the grade.

Safety considerations may justify the addition of a climbing lane regardless of grade or traffic volumes.

6.4.1.2 Multilane Highways

A truck-climbing lane may be considered on a multilane highway if the following conditions are satisfied:

- the directional service volume exceeds 1000 vehicles per hour per lane; <u>and</u>
- the construction costs and the construction impacts (e.g., environmental, right of way) are considered reasonable; <u>and</u>
- one of the following conditions exists:

+ the critical length of grade is exceeded for the 10 mile-per-hour speed reduction curve (see Figure 6.3-A or Figure 6.3-B); <u>or</u>

+ the LOS on the upgrade is E or F; <u>or</u>

+ there is a reduction of one or more LOS when moving from the approach segment to the grade.

In addition, safety considerations may justify the addition of a climbing lane regardless of grade or traffic volumes.

6.4.2 Capacity Analysis

See the *Highway Capacity Manual* for details on how to prepare a capacity analysis for climbing lanes on two-lane and multilane highways.

6.4.3 Design Guidelines

Figure 6.4-A summarizes the design criteria for a truck-climbing lane. In addition, consider the following:

1. <u>Design Speed</u>. For entering speeds equal to or greater than 70 miles per hour, use 70 miles per hour for the truck design speed. For speeds less than 70 miles per hour, use the roadway design speed or the posted speed limit, whichever is less. Under restricted conditions, the designer may want to consider the effect a momentum grade will have on the entering speed. See Section 6.3.3 for additional information on momentum grades. However, the maximum speed will be 70 miles per hour.

2. <u>Superelevation</u>. For horizontal curves, provide superelevation on the truck-climbing lane at the same rate as the adjacent travel lane.

3. <u>Performance Curves</u>. Figure 6.4-B presents the deceleration and acceleration rates for a 200 pound per horsepower truck.

4. <u>End of Full-Width Lane</u>. In addition to the criteria in Figure 6.4-A, ensure there is sufficient sight distance available to the point where the truck, RV or bus will begin to merge into the through travel lane. At a minimum, this will be the stopping sight distance. Desirably, the driver should have decision sight distance available to the end of the taper. See Section 4.3 for the decision sight distance values.

   The full-lane width should be extended beyond the crest vertical curve and not end just beyond the crest vertical curve. Desirably, the full-lane width should not end on a horizontal curve.

5. <u>Signing and Pavement Markings</u>. Contact the Traffic Engineering Division for guidance on signing and pavement markings for climbing lanes.

6.4.4 Downgrades

Truck lanes on downgrades are typically not considered. However, steep downhill grades may also have a detrimental effect on the capacity and safety of facilities with high-traffic volumes and numerous heavy trucks. Although specific criteria have not been established for these conditions, trucks descending steep downgrades in low gear may produce nearly as great an effect on operations as an equivalent upgrade. The need for a truck lane for downhill traffic will be considered on a site-by-site basis.

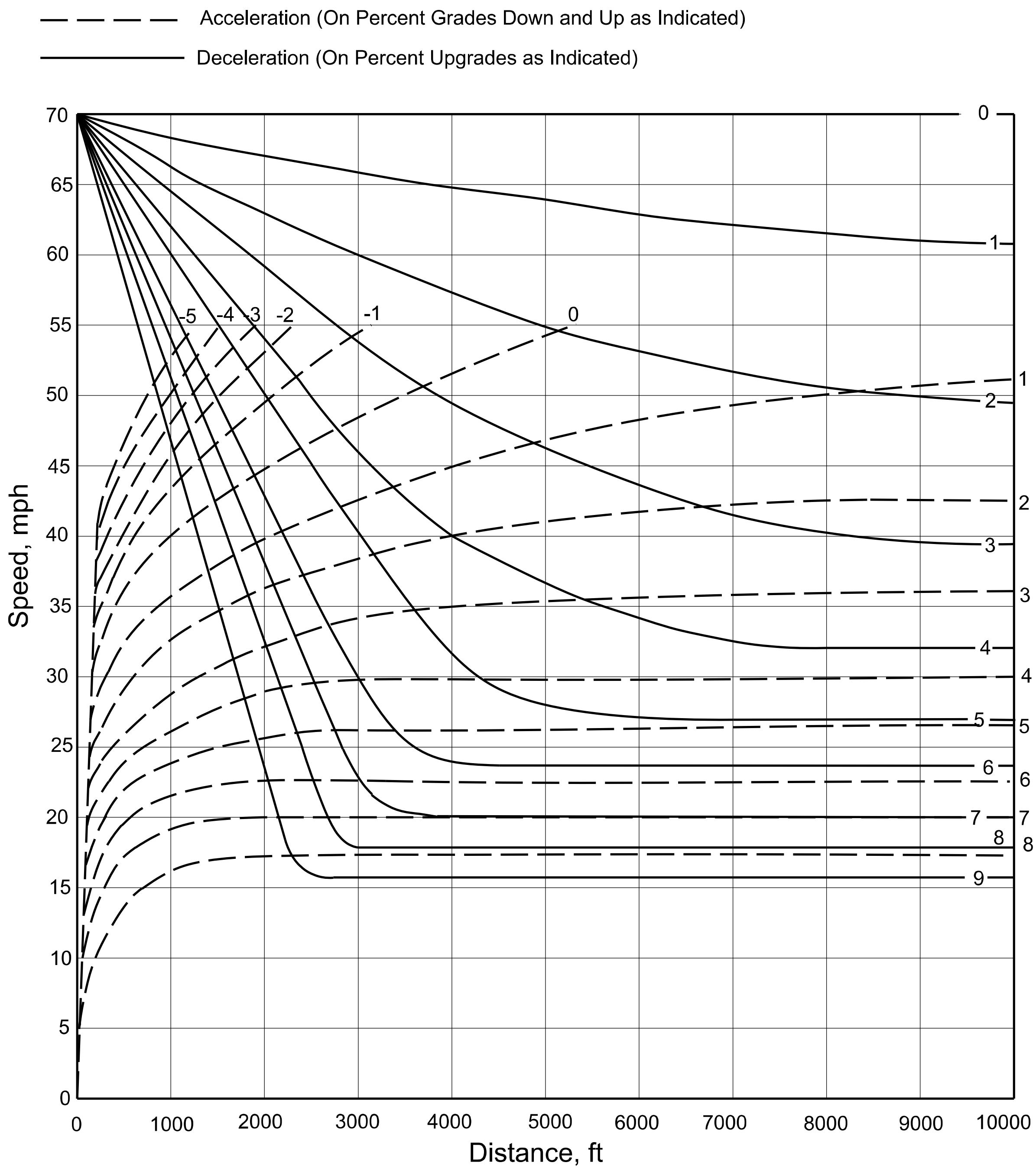
Design Element	Desirable	Minimum
Lane Width	12 ft	Freeway: 12 ft Other Facilities: 11 ft
Shoulder Width	Same as Approach Roadway	6 ft
Cross Slope on Tangent	2%	2%
Beginning of Full-Width Lane(1)	Location where the truck speed has been reduced to 10 mph below the posted speed limit.	Location where the truck speed has been reduced to 45 mph.
End of Full-Width Lane(2)	Location where truck has reached highway posted speed or 55 mph, whichever is less.	Location where truck has reached 10 mph below highway posted speed limit.
Entering Taper	300 ft	300 ft
Exiting Taper	600 ft	50:1
Minimum Full-Width Length	1000 ft or greater	1000 ft

Notes:

1. *Use Figure 6.4-B to determine truck deceleration rates. In determining the applicable truck speed, the designer may consider the effect of momentum grades.*

2. *Use Figure 6.4-B to determine truck acceleration rates. Also, see Comment 4 in Section 6.4.3.*

DESIGN CRITERIA FOR TRUCK-CLIMBING LANES
Figure 6.4-A



Note: For entering speeds equal to or greater than 70 miles per hour, use an initial speed of 70 miles per hour. For speeds less than 70 miles per hour, use the design speed or posted speed limit as the initial speed.

PERFORMANCE CURVES FOR TRUCKS
(200 lb/hp)
Figure 6.4-B

6.4.5 Truck Speed Profile

For highways with a single grade, the critical length of grade and deceleration and acceleration rates can be directly determined from Figure 6.4-B. However, it is often necessary to find the impact of a series of significant grades in succession. If several different grades are present, then a speed profile may need to be developed.

The following example illustrates how to construct a truck speed profile and how to use Figure 6.4-B.

* * * * * * * * * *

Example 6.4-1

Given: Level Approach
G_1 = +3 percent for 800 feet (VPI to VPI)
G_2 = +5 percent for 3200 feet (VPI to VPI)
G_3 = –2 percent beyond the composite upgrade (G_1 and G_2)
V = 60 mile-per-hour design speed with a 55 mile-per-hour posted speed limit
Rural Principal Arterial

Problem: Using the criteria in Figure 6.4-A and Figure 6.4-B, construct a truck speed profile and determine the beginning and ending points of the full-width climbing lane.

Solution: The following steps apply:

Step 1: Determine the truck speed on G_1 using Figure 6.4-B and plot the truck speed at 200-foot increments in Figure 6.4-C. Assume an initial truck speed of 55 miles per hour. Move horizontally along the 55 mile-per-hour line to the 3 percent deceleration curve. This is approximately 2800 feet along the horizontal axis. This is the starting point for G_1.

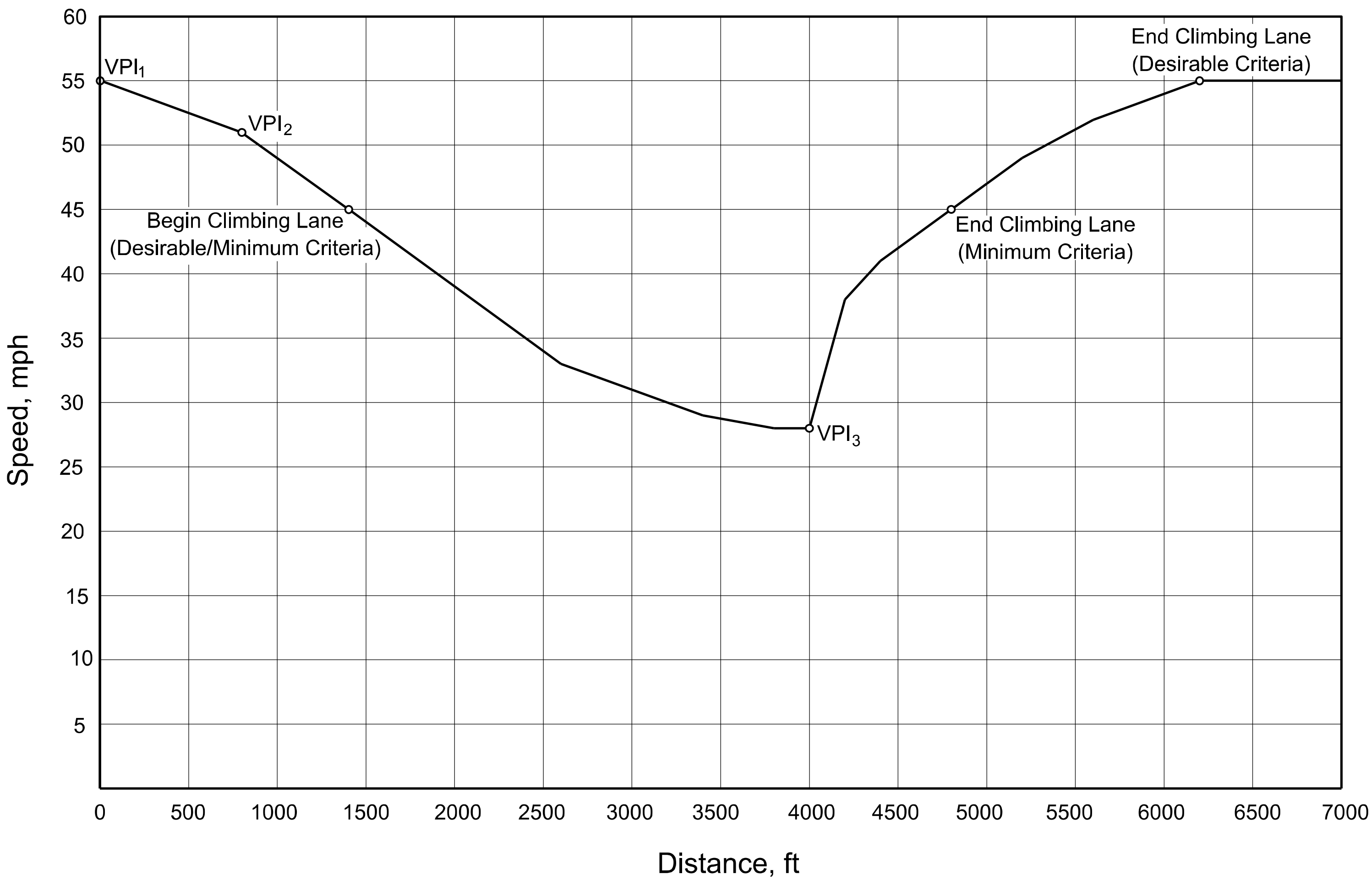
Distance from VPI_1 (feet)	Horizontal Distance on Figure 6.4-B (feet)	Truck Speed (miles per hour)	Comments
0	2800	55	VPI_1
200	3000	54	
400	3200	53	
600	3400	52	
800	3600	51	VPI_2

Step 2: Determine the truck speed on G_2 using Figure 6.4-B and plot the truck speed at 200-foot increments in Figure 6.4-C. From Step 1, the initial speed on G_2 is the final speed from G_1 (i.e., 51 miles per hour). Move right horizontally along the 51 mile-per-hour line to the 5 percent deceleration curve. This is approximately 1900 feet along the horizontal axis. This is the starting point for G_2.

Figure 6.4-C Distance from VPI_1 (feet)	Horizontal Distance on Figure 6.4-B (feet)	Truck Speed (miles per hour)	Comments
800	1900	51	VPI_2
1000	2100	49	
1200	2300	47	
1400	2500	45	
1600	2700	43	
1800	2900	41	
2000	3100	39	
2200	3300	37	
2400	3500	35	
2600	3700	33	
2800	3900	32	
3000	4100	31	
3200	4300	30	
3400	4500	29	
3600	4700	29	
3800	4900	28	
4000	5100	28	VPI_3

Step 3: Determine the truck speed on G_3 using Figure 6.4-B until the truck has fully accelerated to 55 miles per hour and plot the truck speed at 200-foot increments in Figure 6.4-C. The truck will have a speed of 28 miles per hour as it enters the 2 percent downgrade at VPI_3. Read into Figure 6.4-B at the 28 mile-per-hour point on the vertical axis and move over horizontally to the −2 percent line. This is approximately 150 feet along the horizontal axis. This is the starting point for G_3.

Figure 6.4-C Distance from VPI_1 (feet)	Horizontal Distance on Figure 6.4-B (feet)	Truck Speed (miles per hour)	Comments
4000	150	28	VPI_3
4200	350	38	
4400	550	41	
4600	750	43	
4800	950	45	
5000	1150	47	
5200	1350	49	
5400	1550	50	
5600	1750	52	
5800	1950	53	
6000	2150	54	
6200	2350	55	



TRUCK SPEED PROFILE
(Example 6.4-1)
Figure 6.4-C

Step 4: Determine the beginning and end of the full-width climbing lane. From Figure 6.4-A, the desirable and minimum beginning of the full-width lane will be where the truck has reached a speed of 45 miles per hour (10 miles per hour below the posted speed). This point occurs 1400 feet beyond VPI_1.

For ending the full-width climbing lane, the desirable criteria from Figure 6.4-A is where the truck speed has reached the posted speed limit (55 miles per hour) or 6200 feet beyond the VPI_1. The minimum criteria is where the truck has reached a speed of 45 miles per hour (10 miles per hour below the posted speed). This occurs at 4800 feet beyond VPI_1.

6.5 VERTICAL CURVES

6.5.1 Crest Vertical Curves

6.5.1.1 Equations

Crest vertical curves are in the shape of a parabola. The basic equations for determining the minimum length of a crest vertical curve are:

$$L = \frac{AS^2}{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2} \qquad \text{(Equation 6.5-1)}$$

$$K = \frac{S^2}{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2} \qquad \text{(Equation 6.5-2)}$$

$$L = KA \qquad \text{(Equation 6.5-3)}$$

Where:

- L = length of vertical curve, feet
- A = absolute value of the algebraic difference between the two tangent grades, percent
- S = sight distance (SSD, DSD, PSD), feet
- h_1 = height of eye above road surface, feet
- h_2 = height of object above road surface, feet
- K = horizontal distance needed to produce a 1 percent change in gradient

The length of a crest vertical curve will depend upon "A," the absolute value of the algebraic difference between the two tangent grades, for the specific curve and upon the selected sight distance, height of eye and height of object. Equation 6.5-1 and the resultant values of K are predicated on the sight distance being less than the length of vertical curve. However, these values can also be used, without significant error, where the sight distance is greater than the length of vertical curve. The following sections discuss the selection of K-values.

6.5.1.2 Stopping Sight Distance

The principal control in the design of crest vertical curves is to ensure that stopping sight distance (SSD) is available throughout the vertical curve. The following discusses the application of K-values for various operational conditions:

1. Passenger Cars (Level Grade). Figure 6.5-A presents K-values for passenger cars on a level grade. Level conditions are assumed where the grade on the far side of the vertical curve is less than 3 percent. The minimum values are calculated by assuming h_1 = 3.5 feet, h_2 = 2 feet and S = SSD in the basic equation for crest vertical curves (Equation 6.5-2).

Design Speed (miles per hour)	Stopping[1] Sight Distance (feet)	Rate of Vertical Curvature, K-Value[2]	
		Minimum[3]	
		Calculated	Design
15	80	3.0	3
20	115	6.1	7
25	155	11.1	12
30	200	18.5	19
35	250	29.0	29
40	305	43.1	44
45	360	60.1	61
50	425	83.7	84
55	495	113.5	114
60	570	150.6	151
65	645	192.8	193
70	730	246.9	247
75	820	311.6	312
80	910	383.7	384

Notes:

1. *Stopping sight distances (SSD) are from Figure 4.1-A.*
2. *Maximum K-value for drainage on curbed roadways is 167; see Section 6.5.1.4.*
3. *(Minimum)* $K = \frac{SSD^2}{2158}$*, where:* $h_1 = 3.5$ *feet,* $h_2 = 2$ *feet*

K-VALUES FOR CREST VERTICAL CURVES — STOPPING SIGHT DISTANCES
(Passenger Cars — Level Grades)
Figure 6.5-A

2. Passenger Cars (Grade Adjusted). For crest vertical curves, consider grade adjustments where the downgrade is 3 percent or greater. Where practical, provide stopping sight distances greater than the design values in Figure 4.1-A where horizontal sight restrictions occur on downgrades, even when the horizontal sight obstruction is a cut slope. No adjustment is necessary for grades less than 3 percent or for upgrades. Use Equation 6.5-1 and the grade adjusted SSD from Figure 4.1-C to determine the length of vertical curve.

3. Minimum Length of Curve. The minimum length of a crest vertical curve in feet should be L=3V, where V is the design speed in miles per hour.

4. Minimum Values. Only use minimum K-values where the use of higher value will result in unacceptable social, economic or environmental impacts.

6.5.1.3 Passing Sight Distance

At some locations, it is desirable to provide passing sight distance in the design of crest vertical curves. Section 4.2 discusses the application and design values for passing sight distance on two-lane, two-way highways. Passing sight distance values are used as the “S” value in the basic equation for crest vertical curves (Equation 6.5-1). In addition, the following will apply:

1. Height of Eye (h_1). For passenger cars, h_1 = 3.5 feet.

2. Height of Object (h_2). Passing sight distance is predicated upon the passing driver being able to see a sufficient portion of the top of the oncoming car. Therefore, h_2 = 3.5 feet.

3. K-Values. Figure 6.5-B presents the K-values for passenger cars using the passing sight distances presented in Figure 4.2-A.

6.5.1.4 Drainage

Proper drainage must be considered in the design of crest vertical curves. Typically, drainage problems will not be experienced if the vertical curvature is sharp enough so that a minimum longitudinal gradient of at least 0.3 percent is reached at a point about 50 feet from either side of the apex. To ensure that this objective is achieved, determine the length of the crest vertical curve assuming a K-value of 167 or less. Where the maximum drainage K-value is exceeded, carefully evaluate the drainage design near the apex. With the use of proper cross slopes, drainage generally should not be a problem on crest vertical curves.

Design Speed (miles per hour)	Passing Sight Distance[1] (feet)	Rate of Vertical Curvature[2], K-Value
20	400	57
25	450	72
30	500	89
35	550	108
40	600	129
45	700	175
50	800	229
55	900	289
60	1000	357
65	1100	432
70	1200	514
75	1300	604

Notes:

1. *Design passing sight distances (PSD) are from Section 4.2.*

2. $K = \frac{PSD^2}{2800}$, *where:* $h_1 = 3.5$ *feet,* $h_2 = 3.5$ *feet*

K-VALUES FOR CREST VERTICAL CURVES — PASSING SIGHT DISTANCES
(Passenger Cars)
Figure 6.5-B

6.5.2 Sag Vertical Curves

6.5.2.1 Equations

Sag vertical curves are in the shape of a parabola. Typically, they are designed to allow the vehicular headlights to illuminate the roadway surface (i.e., the height of object = 0.0 feet for a given distance “S.” The light beam from the headlights is assumed to have a 1 degree upward divergence from the longitudinal axis of the vehicle. These assumptions yield the following basic equations for determining the minimum length of sag vertical curves:

$$L = \frac{AS^2}{200[h_3 + S(\tan 1°)]} = \frac{AS^2}{200h_3 + 3.5S} \quad \text{(Equation 6.5-4)}$$

$$K = \frac{S^2}{200h_3 + 3.5S} \quad \text{(Equation 6.5-5)}$$

$$L = KA \quad \text{(Equation 6.5-6)}$$

Where:

- L = length of vertical curve, feet
- A = absolute value of the algebraic difference between the two tangent grades, percent
- S = sight distance (SSD, DSD), feet
- h_3 = height of headlights above pavement surface, feet
- K = horizontal distance needed to produce a 1 percent change in gradient

The length of a sag vertical curve will depend upon “A,” the absolute value of the algebraic difference between the two tangent grades, for the specific curve and upon the selected sight distance and headlight height. Equation 6.5-4 and the resultant values of K are predicated on the sight distance being less than the length of vertical curve. However, these values can also be used, without significant error, where the sight distance is greater than the length of vertical curve. The following sections discuss the selection of K-values.

6.5.2.2 Stopping Sight Distance

The principal control in the design of sag vertical curves is to ensure minimum stopping sight distance (SSD) is available for headlight illumination throughout the sag vertical curve. The following discusses the application of K-values for various operational conditions:

1. Passenger Cars (Level Grade). Figure 6.5-C presents K-values for passenger cars. These are calculated by assuming h_3 = 2 feet and S = SSD in the basic equation for sag vertical curves (Equation 6.5-5). The minimum values represent the lowest acceptable sight distance on a facility. Use longer than the minimum lengths of curves to provide a more aesthetically pleasing design.

2. Passenger Cars (Grade Adjusted). For sag vertical curves, consider grade adjustments where the downgrade is 3 percent or greater. No adjustment is necessary for grades less than 3 percent or for upgrades. Where practical, provide stopping sight distances greater than the design values in Figure 4.1-A where horizontal sight restrictions occur on downgrades, even when the horizontal sight obstruction is a cut slope. Use Equation 6.5-4 and the grade adjusted SSD from Figure 4.1-C to determine the length of vertical curve.

3. Minimum Length. The minimum length of a sag vertical curve in feet should be L=3V, where V is the design speed in miles per hour.

One exception to the minimum length on sag vertical curves applies in curbed sections and on bridges. If the sag is in a low point, the use of the long vertical curves may produce longitudinal slopes too flat to drain stormwater without ponding. For additional guidance, see Section 6.5.2.5.

4. Minimum Values. Only use minimum K-values where the use of higher value will result in unacceptable social, economic or environmental impacts.

Design Speed (miles per hour)	Stopping[1] Sight Distance (feet)	Rate of Vertical Curvature[3], K-Value	
		Calculated	Design
15	80	9.4	10
20	115	16.5	17
25	155	25.5	26
30	200	36.4	37
35	250	49.0	49
40	305	63.4	64
45	360	78.1	79
50	425	95.7	96
55	495	114.9	115
60	570	135.7	136
65	645	156.5	157
70	730	180.3	181
75	820	205.6	206
80	910	231.0	231

Notes:

1. *Stopping sight distances (SSD) are from Figure 4.1-A.*
2. *Maximum K-value for drainage on curbed roadways and bridges is 167, see Section 6.5.2.5.*
3. $K = \dfrac{SSD^2}{400 + 3.5SSD}$, *where:* $h_3 = 2$ *feet*

K-VALUES FOR SAG VERTICAL CURVES — STOPPING SIGHT DISTANCES
(Passenger Cars — Level Grades)
Figure 6.5-C

6.5.2.3 Comfort Criteria

The comfort criteria is based on the effect of change in the vertical direction of a sag vertical curve due to the combined gravitational and centrifugal forces. The general consensus is that riding is comfortable on sag vertical curves when the centripetal acceleration does not exceed 1 foot per second^2. The length-of-curve equation for the comfort criteria is:

$$L = \frac{AV^2}{46.5}$$ (Equation 6.5-7)

Where:

- L = length of vertical curve, feet
- A = absolute value of the algebraic difference between the two tangent grades, percent
- V = design speed, miles per hour

The length of vertical curve needed to satisfy the comfort factor at the various design speeds is only about 50 percent of that needed to satisfy the headlight sight distance criterion for the normal range of design conditions; therefore, comfort criteria may only be applied on fully lighted roadways.

6.5.2.4 Underpasses

Check sag vertical curves through underpasses to ensure that the underpass structure does not obstruct the driver's visibility. Use the following equation to check sag vertical curves through underpasses:

$$L = \frac{AS^2}{800(C - 4.25)}$$ (Equation 6.5-8)

Where:

- L = length of vertical curve, feet
- A = absolute value of the algebraic difference between the two tangent grades, percent
- S = sight distance, feet
- C = vertical clearance of underpass, feet

Compare the L calculated from Equation 6.5-8 for underpasses with the L calculated based on headlight illumination (Equation 6.5-4). The larger of the two lengths will govern.

6.5.2.5 Drainage

Proper drainage must be considered in the design of sag vertical curves on highways with curbs, bridges and medians with concrete median barriers. Drainage problems are minimized if the sag vertical curve is sharp enough so that a minimum longitudinal gradient of at least 0.3 percent is reached at a point about 50 feet from either side of the low point. This results in a

K-value of 167 or less. For most design speeds, the K-values are less than 167; see Figure 6.5-C. However, for higher design speeds and/or where longer sag vertical curves are required on highways with curbs or on bridges, it may be necessary to install additional inlets on either side of the low point.

For a highway without curbs, drainage should not be a problem at sag vertical curves if the highway has proper cross slopes.

6.5.3 Vertical Curve Computations

The following will apply to the mathematical design of vertical curves:

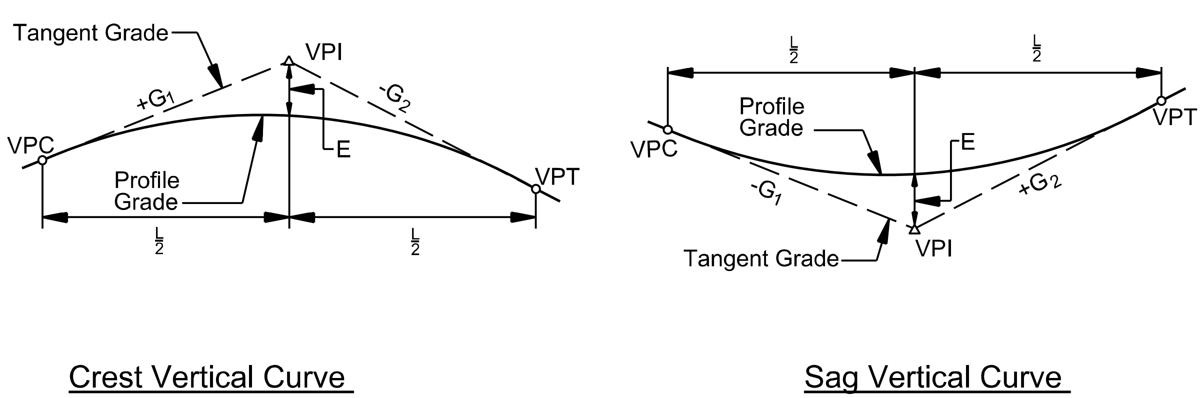
1. Definitions. Figure 6.5-D presents the common terms and definitions used in vertical curve computations.

2. Measurements. All measurements for vertical curves are made on the horizontal or vertical plane, not along the profile grade line. With the simple parabolic curve, the vertical offsets from the tangent vary as the square of the horizontal distance from the VPC or VPT. Elevations along the curve are calculated as proportions of the vertical offset at the point of vertical intersection (VPI). The equations for computing a symmetrical vertical curve are shown in Figure 6.5-E. Figure 6.5-F provides an example of how to use these formulas.

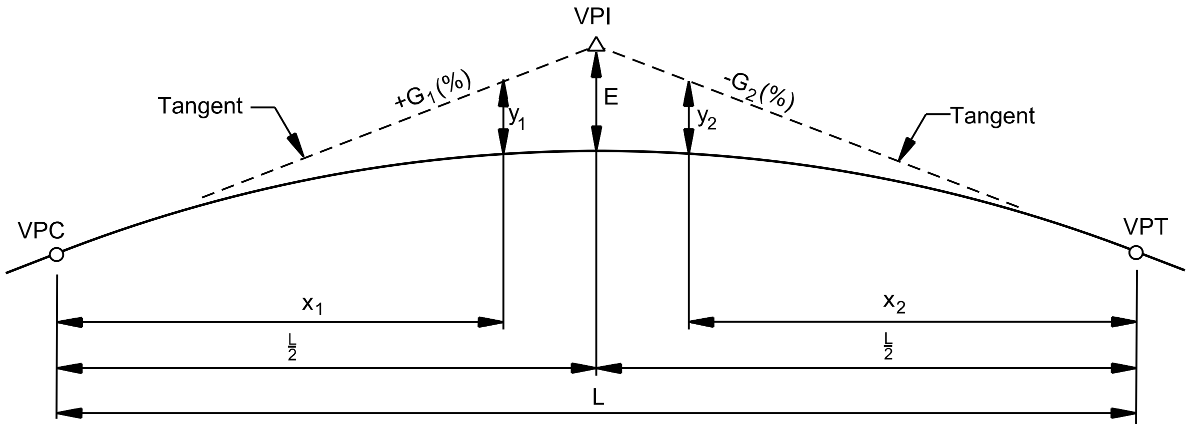
3. Vertical Curve Through Fixed Point. The vertical curve of a highway often must be designed to pass through an established elevation and location. For example, it may be necessary to tie into an existing side road or to clear existing structures. Figure 6.5-G provides the procedure for determining how to pass a vertical curve through a fixed point. Figure 6.5-H and 6.5-I illustrate examples on how to use these formulas.

4. VPI Stationing. The designer may need to determine the VPI station between two known VPIs. Figure 6.5-J illustrates how to determine the intermediate VPI given the gradients, stations and elevations of the other VPIs.

Element	Abbreviation	Definition
Vertical Point of Curvature	VPC	The point at which a tangent grade ends and the vertical curve begins.
Vertical Point of Tangency	VPT	The point at which the vertical curve ends and the tangent grade begins.
Vertical Point of Intersection	VPI	The point where the extension of two tangent grades intersect.
Grade	G_1, G_2	The slope between two adjacent VPIs expressed as a percent. The numerical value for percent of grade is the vertical rise or fall in feet for each 100 feet of horizontal distance. Upgrades in the direction of stationing are identified as positive (+). Downgrades are identified as negative (−).
External Distance	E	The vertical distance (offset) between the VPI and the roadway surface along the vertical curve.
Algebraic Difference in Grade (Absolute Value)	A	The value of A is determined by the difference in percent between two tangent grades ($G_2 - G_1$).
Length of Vertical Curve	L	The horizontal distance in feet from the VPC to the VPT.
Tangent Elevation	Tan. Elev.	The elevation on the tangent line between the VPC and VPI and the VPI and VPT.
Elevation on Vertical Curve	Curve Elev.	The elevation of the vertical curve at any given point along the curve.
Horizontal Distance	x	Horizontal distance measured from the VPC or VPT to any point on the vertical curve in feet.
Tangent Offset	y	Vertical distance from the tangent line to any point on the vertical curve in feet.
Low/High Point	x_T	The station at the high point for crest curves or the low point for sag curves. At this point, the slope of the tangent to the curve is equal to 0 percent.
Symmetrical Curve	—	The VPI is located at the mid-point between VPC and VPT stationing.



VERTICAL CURVE DEFINITIONS
Figure 6.5-D



E	=	External distance at VPI, feet
y	=	Any tangent offset, feet
L	=	Horizontal length of vertical curve, feet
x	=	Horizontal distance from VPC or VPT to any ordinate "y," feet
G_1 & G_2	=	Rates of grade, expressed algebraically, percent

Note: All expressions are to be calculated algebraically.
(Use algebraic signs of grades; grades in percent.)

1. Elevations of VPC and VPI:

$$\text{VPC ELEV.} = \text{VPI ELEV.} - \left(\frac{G_1}{100} \text{ x } \frac{L}{2}\right) \qquad \text{(Equation 6.5-9)}$$

$$\text{VPT ELEV.} = \text{VPI.} + \left(\frac{G_2}{100} \text{ x } \frac{L}{2}\right) \qquad \text{(Equation 6.5-10)}$$

2. For the elevation of any point "x" on a vertical curve:

$$\text{CURVE ELEV.} = \text{TAN ELEV.} \pm y$$

Where:

Left of VPI (x_1 measured from VPC):

(a) $$\text{TAN ELEV.} = \text{VPC ELEV.} + \left(\frac{G_1}{100}\right) x_1 \qquad \text{(Equation 6.5-12)}$$

(b) $$y_1 = x_1^2 \frac{(G_2 - G_1)}{200\ L} \qquad \text{(Equation 6.5-13)}$$

SYMMETRICAL VERTICAL CURVE EQUATIONS
Figure 6.5-E

Right of VPI (x_2 measured from VPT):

(a) $\text{TAN ELEV.} = \text{VPT ELEV.} - \left(\frac{G_2}{100}\right) x_2$ (Equation 6.5-14)

(b) $y_2 = {x_2}^2 \frac{(G_2 - G_1)}{200\ L}$ (Equation 6.5-15)

At the VPI:

$y = E$ and $x = L/2$

(a) $\text{TAN ELEV.} = \text{VPC ELEV.} + \frac{G_1 L}{200}$

or $\text{TAN ELEV.} = \text{VPT ELEV.} - \frac{G_2 L}{200}$ (Equation 6.5-16)

(b) $E = \frac{L(G_2 - G_1)}{800}$ (Equation 6.5-17)

3. Calculating high or low point in the vertical curve:

(a) To determine distance "x_T" from VPC: $x_T = \frac{L\ G_1}{G_1 - G_2}$ (Equation 6.5-18)

(b) To determine high or low point stationing: VPC STA. + x_T (Equation 6.5-19)

(c) To determine high or low point elevation on a vertical curve:

$\text{ELEV.}_{\text{HIGH OR LOW POINT}} = \text{VPC ELEV.} - \frac{L\ G_1^2}{(G_2 - G_1)\ 200}$ (Equation 6.5-20)

SYMMETRICAL VERTICAL CURVE EQUATIONS
Figure 6.5-E
(Continued)

* * * * * * * * * *

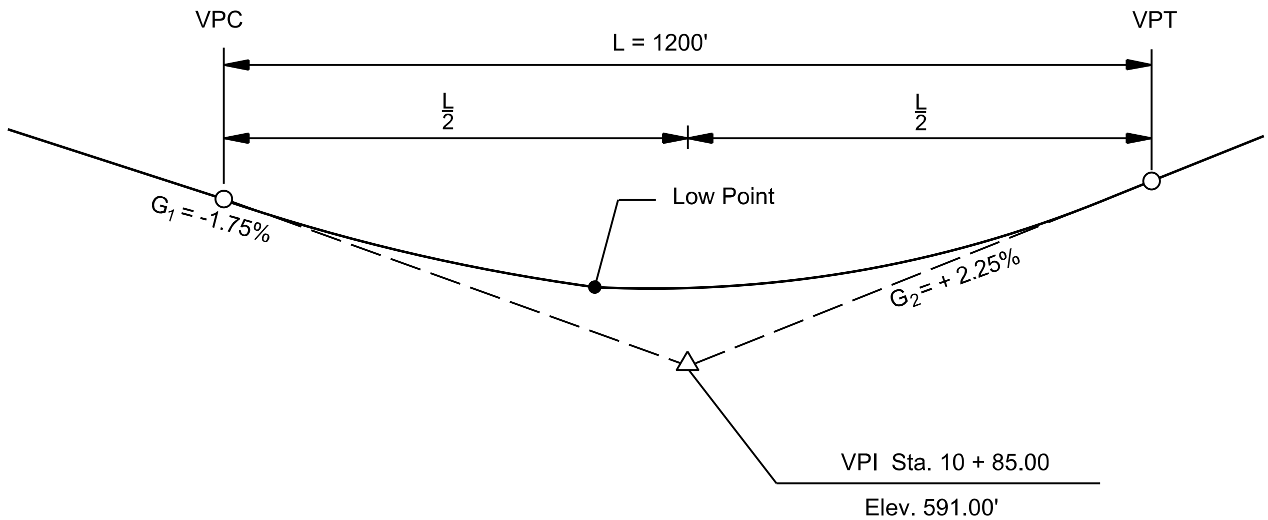
Example 6.5-1

Given: G_1 = −1.75 percent
G_2 = +2.25 percent
Elev. of VPI = 591.00 feet
Station of VPI = 10 + 85.00
L = 1200 feet
Symmetrical Vertical Curve
Rural Area

Problem: Compute the vertical curve elevations for each 100-foot station. Compute the low point elevation and stationing.

Solution:

1. Draw a diagram of the vertical curve and determine the stationing at the beginning (VPC) and the end (VPT) of the curve.



VPC Station = VPI Sta − ½L = (10 + 85) − 600 = 4 + 85.00
VPT Station = VPI Sta + ½L = (10 + 85) + 600 = 16 + 85.00

2. Elevations of VPC and VPI:

$$\text{VPC ELEV.} = 591.00 - \left(\frac{-1.75}{100} \text{ x } \frac{1200}{2}\right) = 601.50 \text{ feet} \qquad \text{(Equation 6.5-21)}$$

$$\text{VPT ELEV.} = 591.00 + \left(\frac{2.25}{100} \text{ x } \frac{1200}{2}\right) = 604.50 \text{ feet} \qquad \text{(Equation 6.5-22)}$$

3. Set up a table to show the vertical curve elevations at the 100-foot stations, substituting the values into Equations 6.5-12 through 6.5-15. Calculate the elevation to the nearest 0.01 foot.

SYMMETRICAL VERTICAL CURVE COMPUTATIONS
(Example 6.5-1)
Figure 6.5-F

Example 6.5-1 (continued)

Solution:

Station	Control Point	Tangent Elevation (feet)	x	x^2	$y = x^2/60{,}000$	Grade Elevation (feet)
4+85	VPC	601.50	0	0	0.00	601.50
5+85		599.75	100	10000	0.17	599.92
6+85		598.00	200	40000	0.67	598.67
7+85		596.25	300	90000	1.50	597.75
8+85		594.50	400	160000	2.67	597.17
9+85		592.75	500	250000	4.17	596.92
10+85	VPI	591.00	600	360000	6.00	597.00
11+85		593.25	500	250000	4.17	597.42
12+85		595.50	400	160000	2.67	598.17
13+85		597.75	300	90000	1.50	599.25
14+85		600.00	200	40000	0.67	600.67
15+85		602.25	100	10000	0.17	602.42
16+85	VPT	604.50	0	0	0.00	604.50

4. Calculate the low point using Equations 6.5-18, 6.5-19 and 6.5-20:

$$x_T = \frac{1200\ (-1.75)}{-1.75 - 2.25} = \frac{-2100}{-4.00} = 5 + 25 \text{ ft from VPC}$$

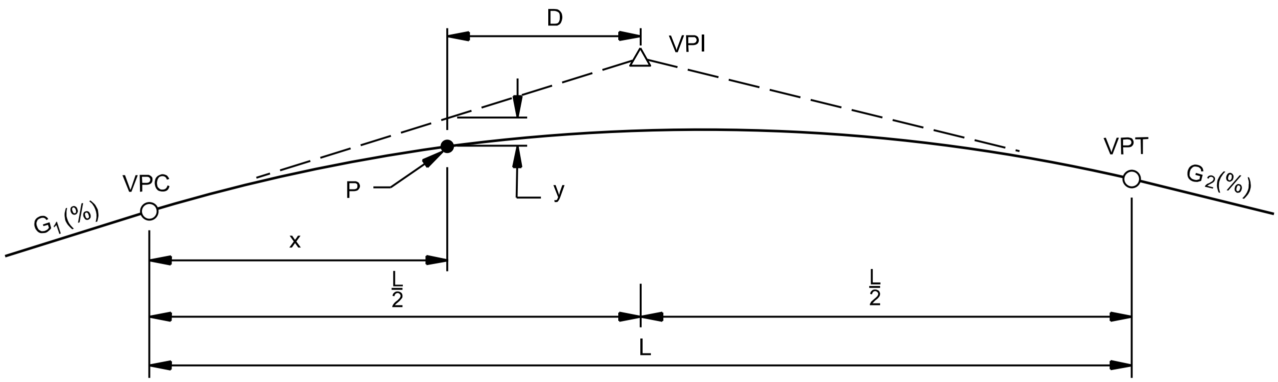
therefore, the Station at the low point is:

$$VPC_{STA} + x_T = (4 + 85) + (5 + 25) = 10 + 10.00$$

Elevation at the low point on curve is:

$$\text{Elevation of low point} = 601.50 - \frac{1200\ (-1.75)^2}{(2.25 - (-1.75))\ 200} = 601.50 - 4.59 = 596.91 \text{ feet}$$

VERTICAL CURVE COMPUTATIONS
(Example 6.5-1)
(Continued)
Figure 6.5-F



G_1 = Grade in, percent
G_2 = Grade out, percent
A = Algebraic difference in grades, percent
y = Vertical curve correction at point P, feet
x = Distance from VPC to P, feet
D = Distance from P to VPI, feet
L = Length of vertical curve, feet

Given: G_1, G_2, D

Problem: Determine the length of a vertical curve required to pass through a given point (P).

Solution:

1. Find algebraic difference in grades:

$$A = G_2 - G_1$$

$$\text{VPT ELEV.} = \text{VPI ELEV.} + \left(\frac{G_2}{100}\right)\left(\frac{L}{2}\right) \qquad \text{(Equation 6.5-23)}$$

2. Find vertical curve correction at Point P:

$$y = x^2 \left(\frac{G_2 - G_1}{200\ L}\right)$$

From Equation 6.5-13 (x measured from VPC)

3. From inspection of the above diagram:

$$x + D = L / 2 \text{ or } L = 2 (x + D) \qquad \text{(Equation 6.5-24)}$$

SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT
Figure 6.5-G

By substituting 2(x + D) for L, and A for ($G_2 - G_1$) into Equation 6.5-13. Yields:

$$A x^2 + (-400\,y)\,x + (-400\,D\,y) = 0 \qquad \text{(Equation 6.5-25)}$$

4. Solve for "x" using the quadratic equation:

$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} = \frac{400y \pm \sqrt{160000\,y^2 + 1600ADy}}{2A} \qquad \text{(Equation 6.5-26)}$$

Solving for "x" will result in two answers. If both answers are positive, there are two solutions. If one answer is negative, it can be eliminated and only one solution exists.

5. Substitute x and D into Equation 6.5-24 and solve for L:

Note: Two positive x values, will result in two L solutions. Desirably, use the longer vertical curve solution provided it meets the sight distance criteria (based on the selected design speed and algebraic difference in grades).

SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT

(Continued)

Figure 6.5-G

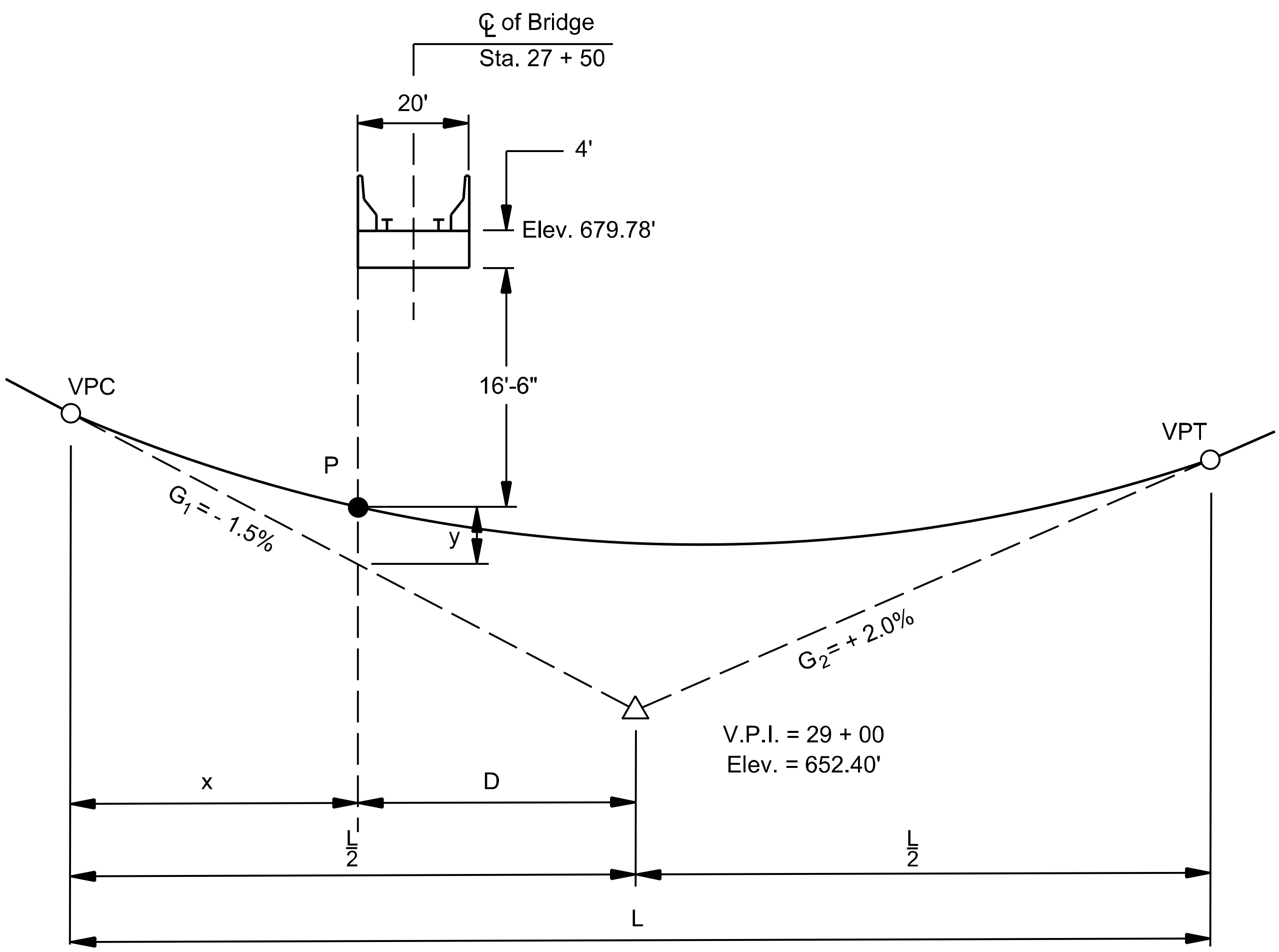
Example 6.5-2

Given: Design Speed = 55 miles per hour
G_1 = − 1.5 percent
G_2 = + 2.0 percent
A = 3.5 percent
VPI Station = 29 + 00.00
VPI Elevation = 652.40 feet

Problem: At Station 27 + 50, a new highway must pass under the center of an existing railroad that is at elevation 679.78 feet at the highway centerline. The railroad bridge that will be constructed over the highway will be 4 feet in depth, 20 feet in width and at right angles to the highway. Determine the length of the symmetrical vertical curve that would be required to provide a 16 feet-6 inches clearance under the railroad bridge.

Solution:

1. Sketch the problem with known information labeled.



SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT
(Example 6.5-2)
Figure 6.5-H

<u>Example 6.5-2</u> (continued)

2. Determine the station where the minimum 16 feet-6 inches vertical clearance will occur (Point P):

   From inspection of the sketch, the critical location is on the left side of the railroad bridge. The critical station is:

   STA. P = BRIDGE CENTERLINE STATION − ½ (BRIDGE WIDTH)
   STA. P = STA. 27 + 50 − ½ (20)
   STA. P = STA. 27 + 40

3. Determine the elevation of Point P:

   ELEV. P = ELEV. TOP RAILROAD BRIDGE − BRIDGE DEPTH − CLEARANCE
   ELEV. P = 679.78 − 4.0 − 16.5
   ELEV. P = 659.28 feet

4. Determine distance, D, from Point P to VPI:

   D = STA. VPI − STA. P = (29 + 00) − (27 + 40) = 160 feet

5. Determine the tangent elevation at Point P:

$$\text{ELEV. TANGENT AT P} = \text{VPI ELEV.} - \left(\frac{G_1}{100}\right)D = 652.40 - \left(\frac{-1.5}{100}\right)160 = 654.80 \text{ feet}$$

6. Determine the vertical curve correction (y) at Point P:

   y = ELEV. ON CURVE − ELEV. OF TANGENT = 659.28 − 654.80 = 4.48 feet

7. Solve for x using Equation 6.5-26:

$$x = \frac{400\,(4.48) \pm \sqrt{(160000)(4.48)^2 + 1600(3.5)(160)(4.48)}}{2(3.5)}$$

   x = 640 feet AND x = −128 feet (Disregard)

SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT
(Example 6.5-2)
(Continued)
Figure 6.5-H

<u>Example 6.5-2</u> (continued)

8. Using Equation 6.5-24, solve for L:

   $L = 2(x + D)$
   $L = 2(640 + 160)$
   $L = 1600$ feet

9. Determine if the solution meets the passenger car stopping sight distance for the 55 mile-per-hour design speed. From Figure 6.5-C, the minimum design K-value:

   $K = 115$

   The algebraic difference in grades:

   $A = G_2 - G_1 = (+2.0) - (-1.5) = 3.5.$

   From Equation 6.5-6, determine the minimum length of vertical curve which meets the stopping sight distance:

   $L_{MIN} = KA$
   $L_{MIN} = (115)\ 3.5 = 402.5$ feet

   $L = 1600$ feet, which exceeds the minimum design stopping sight distance.

SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT
(Example 6.5-2)
(Continued)
Figure 6.5-H

Example 6.5-3

An existing two-lane bridge on a rural arterial over a freeway is being replaced with a four-lane bridge. The existing bridge does not meet current minimum vertical clearance criteria. The new bridge will have four 12-foot lanes with 8-foot outside shoulders.

Given: Freeway: Design Speed = 70 miles per hour
Freeway is on a curve and is fully superelevated at a rate of 8 percent
Grade = +2.0 percent
North Bound Freeway Elevation at Tie Equation = 445.00 feet

Arterial: Design Speed = 60 miles per hour
G_1 = − 3.0 percent
G_2 = − 1.0 percent
A = 2.0 percent
VPI Station = 83 + 00.00
VPI Elevation = 461.20 feet
Bridge Cross Slope = 2.0%

Problem: At Station 80 + 50, the new bridge will pass over the freeway centerline for the northbound roadway, which is at elevation 445.00 feet. The bridge that will be constructed over the highway will be 5 feet in depth and at right angles to the highway. Determine the length of the symmetrical vertical curve that would be required to provide a minimum 17 feet – 0 inches clearance over the entire freeway cross section.

Solution:

1. Sketch the problem with known information labeled. See the figure on the next page.

2. Determine the station where the minimum 17-foot vertical clearance will occur (Point P):

   From inspection of the sketch, the critical location is on the left side of the bridge where the arterial crosses the northbound freeway roadway. The minimum 17-foot vertical clearance must be available over the entire northbound freeway roadway. Because the freeway is superelevated, the critical point will be the outside edge of the freeway right shoulder. The critical station is:

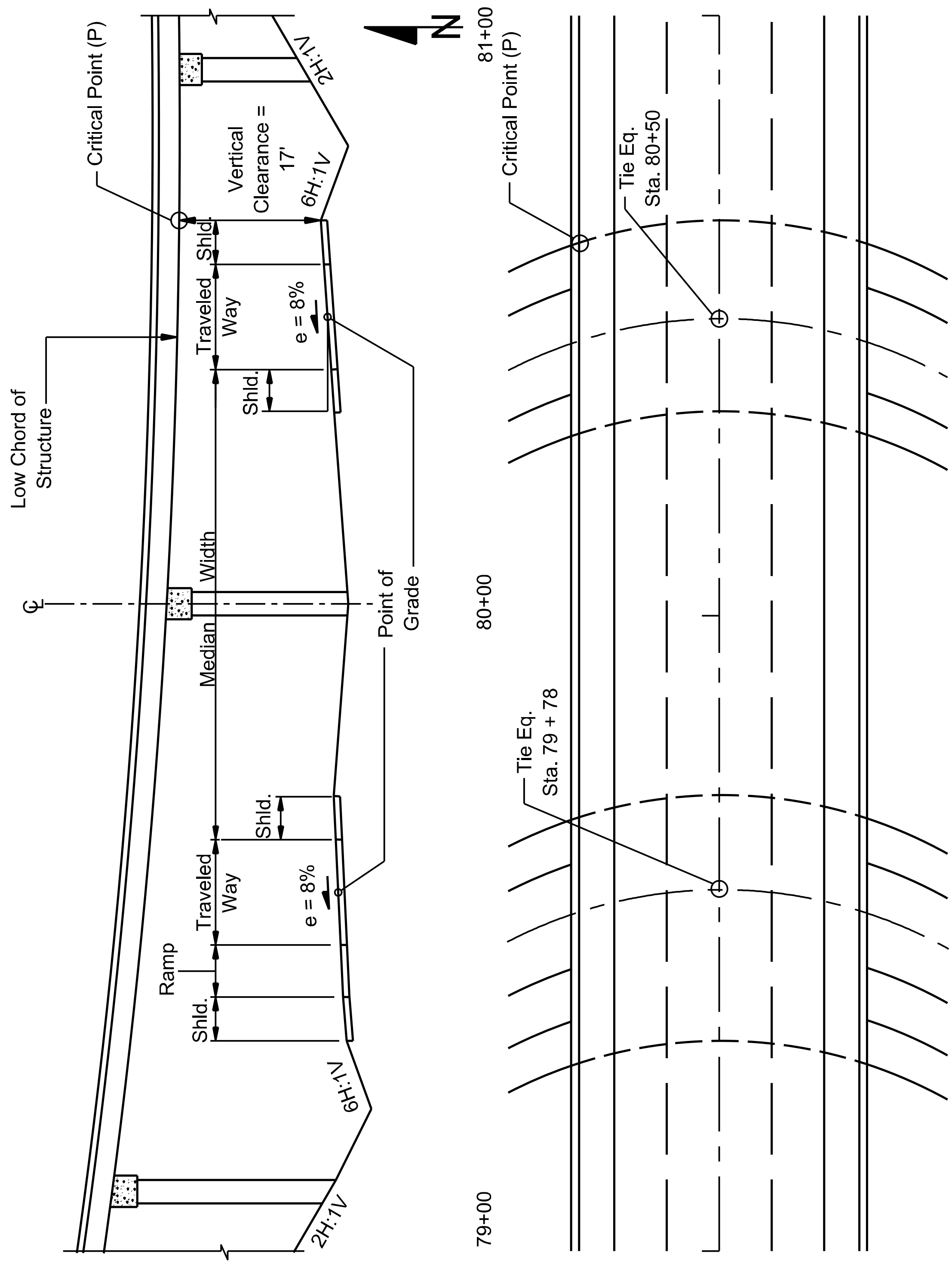
   STA. P = BRIDGE CENTERLINE STATION + (FREEWAY TRAVEL LANE AND SHOULDER WIDTH)
   STA. P = STA. 80 + 50 + (12+12)
   STA. P = STA. 80 + 74

3. Determine the elevation of Point P:

   ELEV. P = ELEV. FREEWAY OUTSIDE SHOULDER AT CRITICAL POINT + BRIDGE DEPTH + CLEARANCE

SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT
(Example 6.5-3)
Figure 6.5-I

<u>Example 6.5-3</u> (continued)



SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT
(Example 6.5-3)
(Continued)
Figure 6.5-I

<u>Example 6.5-3</u> (continued)

To find the elevation at freeway outside shoulder at Point P, first determine the elevation along the freeway centerline to the outside edge of the bridge. Then, calculate the elevation of the outside edge of the shoulder. Note that the total shoulder width includes the 10-foot paved width plus the 2-foot non-paved width.

FREEWAY POINT OF GRADE ELEVATION (outside edge of bridge) = FREEWAY ELEVATION AT THE TIE EQUATION + FREEWAY GRADE x ½ (BRIDGE WIDTH)
ELEV. POG = 445.00 + 0.02 x ½ (64)
ELEV. POG = 445.64 feet

ELEV. AT OUTSIDE SHOULDER = ELEV. POG + e_d (FREEWAY TRAVEL LANE WIDTH + FREEWAY SHOULDER WIDTH)
ELEV. AT OUTSIDE SHOULDER = 445.64 + 0.08 (12+12)
ELEV. AT OUTSIDE SHOULDER = 447.56 feet

ELEV. P = ELEV. FREEWAY OUTSIDE SHOULDER AT CRITICAL POINT + BRIDGE DEPTH + CLEARANCE
ELEV. P = 447.56 + 5.0 + 17.0
ELEV. P = 469.56 feet

4. Determine the ELEV. at the Arterial Profile Grade Line at Point P. The vertical curve on the arterial must pass through this elevation to meet the minimum vertical clearance of 17 feet for the freeway.

   ARTERIAL PGL ELEV. = ELEV. P + (BRIDGE CROSS SLOPE x ½ (BRIDGE WIDTH))
   ARTERIAL PGL ELEV. = 469.56 + (0.02 x ½ (64))
   ARTERIAL PGL ELEV. = 470.20 feet

5. Determine distance, D, from Point P to VPI:

   D = STA. VPI − STA. P = (83 + 00) − (80 + 74) = 226 feet

6. Determine the tangent elevation at Point P:

   $$\text{ELEV. TANGENT AT P} = \text{VPI ELEV.} - \left(\frac{G_1}{100}\right)D$$

   $$\text{ELEV. TANGENT AT P} = 461.20 - \left(\frac{-3.0}{100}\right)226 = 467.98 \text{ feet}$$

7. Determine the vertical curve correction (y) at Point P:

   y = ELEV. ON CURVE − ELEV. OF TANGENT = 470.20 − 467.98 = 2.22 feet

SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT
(Example 6.5-3)
(Continued)
Figure 6.5-I

Example 6.5-3 (continued)

8. Solve for x using Equation 6.5-26:

$$x = \frac{400\,(2.22) \pm \sqrt{(160000)(2.22)^2 + 1600(2.0)(226)(2.22)}}{2(2.0)}$$

x = 609 feet AND x = −164 feet (Disregard)

9. Using Equation 6.5-24, solve for L:

$L = 2(x + D)$
$L = 2(609 + 226)$
$L = 1670$ feet

10. Determine if the solution meets the passenger car stopping sight distance for the 60 mile-per-hour design speed. From Figure 6.5-C, the minimum design K-value:

$K = 136$

The algebraic difference in grades:

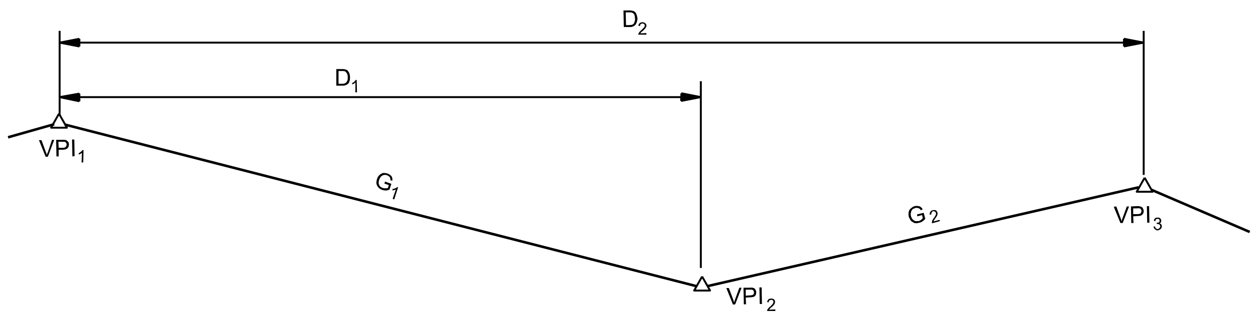
$A = G_2 - G_1 = (-3.0) - (-1.0) = 2.0.$

From Equation 6.5-6, determine the minimum length of vertical curve which meets the stopping sight distance:

$L_{MIN} = KA$
$L_{MIN} = (136)\ 2.0 = 272$ feet

L = 1670 feet, which exceeds the minimum design stopping sight distance.

SYMMETRICAL VERTICAL CURVE THROUGH A GIVEN POINT
(Example 6.5-3)
(Continued)
Figure 6.5-I



D_1 = Distance between VPI_1 and VPI_2, feet
D_2 = Distance between VPI_1 and VPI_3, feet

Given: Station and Elevation at VPI_1
Station and Elevation at VPI_3
G_1, G_2, percent

Problem: Find Station and Elevation of VPI_2.

Solution:

1. Find the station of VPI_2:

$$D_1 = \frac{(\text{ELEV. VPI}_3 - \text{ELEV. VPI}_1) - G_2 D_2}{G_1 - G_2} \qquad \text{(Equation 6.5-27)}$$

$$\text{STA. VPI}_2 = \text{STA.VPI}_1 + D_1$$

2. Find the elevation of VPI_2:

$$\text{ELEV. VPI}_2 = \text{ELEV. VPI}_1 + G_1 D_1 \qquad \text{(Equation 6.5-28)}$$

VERTICAL CURVE COMPUTATION
(Intermediate VPI)
Figure 6.5-J

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6.6 VERTICAL CLEARANCES

Vertical clearance is required above all sections of the roadway surface, including the shoulder. Figure 6.6-A presents the minimum roadway vertical clearances. The clearance must be available over the traveled way, shoulder and any anticipated future widening.

	Freeways	**Arterials**	**Collectors**	**Local Roads and Streets**
New/Replaced Overpassing Bridges*	17 ft – 0 in	17 ft – 0 in	16 ft – 0 in	16 ft – 0 in
Existing Overpassing Bridges	16 ft – 0 in	16 ft – 0 in	16 ft – 0 in	14 ft – 0 in
Pedestrian Bridges	18 ft – 0 in			
Overhead Signs Structures	17 ft – 6 in			
Overhead Utilities	Contact Utilities Office			
Railroads New/Replaced Bridges (Clearance Over)	23 ft – 0 in			
Railroads Widening Existing Bridges (Clearance Over)	Maintain Existing Clearance			
Navigable Water	Contact Environmental Services Office			
Major Lakes and Reservoirs	8 ft – 0 in above the maximum operating pool			
Rivers	2 ft – 0 in above the design high water elevation. Freeboard may be increased up to 7 ft – 0 in for large rivers.			
Tidal Waters	2 ft – 0 in above the 10-yr high water elevation including wave height			

** Table value includes allowance for future overlays.*

MINIMUM VERTICAL CLEARANCES
Figure 6.6-A

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6.7 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. *Highway Capacity Manual 2010*, TRB, 2010.

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Chapter 7

CROSS SECTION ELEMENTS

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Table of Contents

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(Continued)

Chapter 7
CROSS SECTION ELEMENTS

The highway cross section will establish the basic operational and safety features for the facility, and it will have a significant impact on the project cost, especially for right of way. Chapters 14 through 18 contain typical sections and design criteria for cross sections of local roads and streets, collectors, arterials and freeways. This chapter provides guidance in the design of cross section elements, including the roadway section (e.g., travel lanes, shoulders, auxiliary lanes, passing lanes, curbing), roadway elements (e.g., fill and cut slopes, sidewalks), medians, TWLTL, and bridge and underpass cross sections.

7.1 DEFINITIONS/NOMENCLATURE

Figures 7.1-A, 7.1-B, 7.1-C and 7.1-D provide the basic nomenclature for cross section elements for freeways, rural highways, urban streets with curb and gutter, and urban streets with valley gutters, respectively. The following definitions apply to the highway cross section:

1. Auxiliary Lane. The portion of the roadway adjoining the through traveled way for purposes supplementary to through traffic movement including parking, speed change, turning, storage for turning, weaving or truck climbing.

2. Backslope. The side slope created by the connection of the ditch bottom or shelf, upward and outward, to the natural ground.

3. Back Lip. The portion of a valley gutter section beyond the gutter.

4. Buffer. Where used, the area or strip between the roadway and a sidewalk.

5. Catch Curb. The curb type used where the adjacent travel lane or shoulder drains towards the curb and gutter.

6. Cross Slope. The slope in the cross section view of the travel lanes, expressed as a percent or ratio based on the change in horizontal compared to the change in vertical.

7. Depressed Median. A median that is lower in elevation than the traveled way and designed to carry a certain portion of the roadway runoff.

8. Divided Highway. A roadway that has separate traveled ways, usually with a depressed, raised or CMB median for traffic in opposite directions.

9. Fill Slopes. Slopes extending outward and downward from the shoulder hinge point to intersect the natural ground line.

10. Flush Median. A paved median that is essentially level with the surface of the adjacent traveled way.

11. Foreslope. This is the side slope in a cut section created by connecting the shoulder to the ditch hinge point, downward and outward.

12. Hinge Point. The point where the height of fill and depth of cut are determined. For fills, the point is located at the intersection of the shoulder and the fill slope. For cuts, the hinge point is located at the toe of the backslope.

13. Median Slope. The slope in the cross section view of a median beyond the shoulder, expressed as a ratio of the change in horizontal to the change in vertical.

14. Roadway. The combination of the traveled way, both shoulders and/or gutters, and any auxiliary lanes on the mainline highway. Traveled ways separated by a depressed median have two or more roadways.

15. Shelf. On curbed facilities, the relatively flat area located between the back of the curb and the break for the fill or backslopes.

16. Shoulder. The portion of the roadway contiguous to the traveled way for the accommodation of stopped vehicles, for emergency use, and for lateral support of base and surface courses. On sections with curb and gutter, the shoulder includes the gutter.

17. Shoulder Slope. The slope in the cross section view of the shoulders, expressed as a percent or ratio.

18. Slope Offset. On curbed facilities with sidewalks, the distance between the back of the sidewalk and the break for the fill slope or backslope.

19. Sloping Curb. A longitudinal element placed at the roadway edge for delineation, to control drainage, to control access, etc. Sloping curbs have a height of 6 inches or less with a face no steeper than 1 horizontal (H) to 3 vertical (V).

20. Spill Curb. The curb type used where the adjacent travel lane or shoulder drains away from the curb and gutter.

21. Toe of Slope. The intersection of the fill slope or foreslope with the natural ground or ditch bottom.

22. Top of Slope (Cut). The intersection of the backslope with the natural ground.

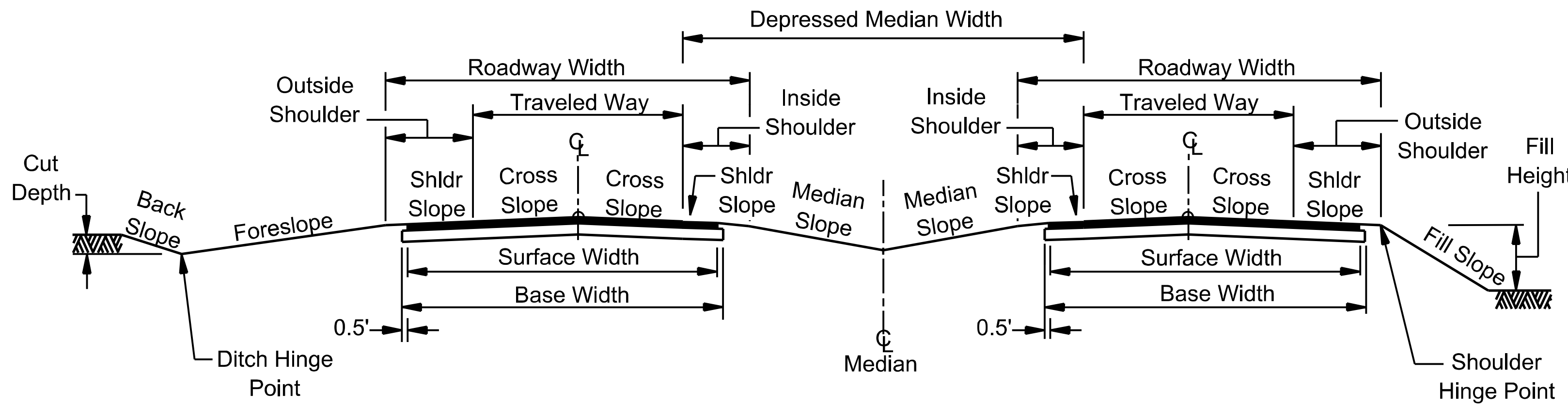
23. Traveled Way. The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

24. Two-Way-Left-Turn-Lane (TWLTL). An area within the flush median marked continuously along a street or highway to provide a deceleration and storage area, out of the through traffic stream, for vehicles traveling in either direction to use in making left turns into and out of access points.

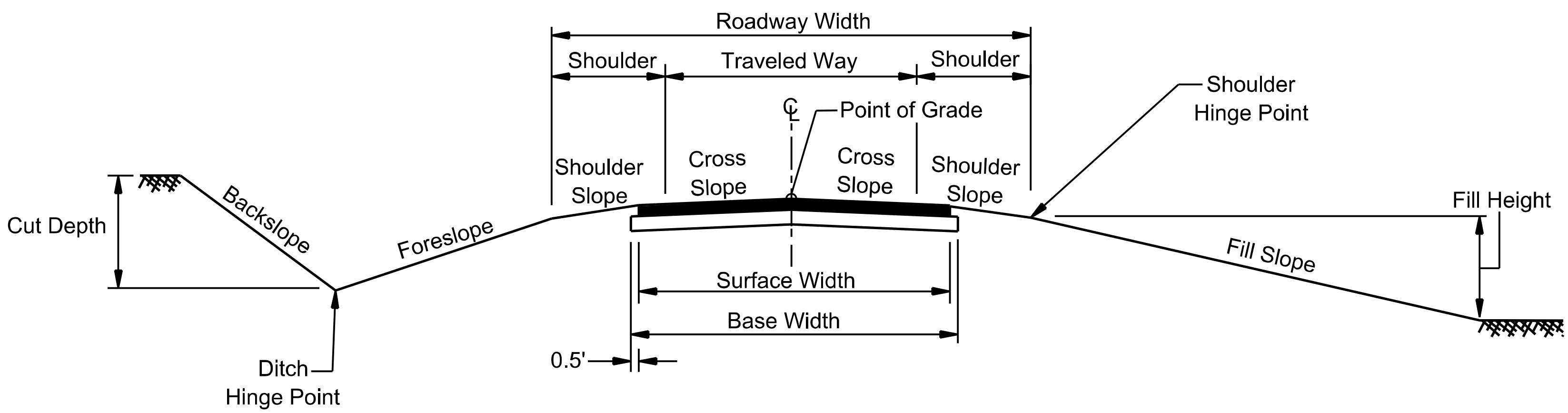
25. Undivided Highway. A roadway that does not have a physical barrier (e.g., depressed median, CMB median) between opposing traffic lanes.

26. Valley Gutter. A paved longitudinal element placed at the roadway edge to control drainage. The valley gutter is designed with a backslope of 10 percent and a width of 3 feet or greater.

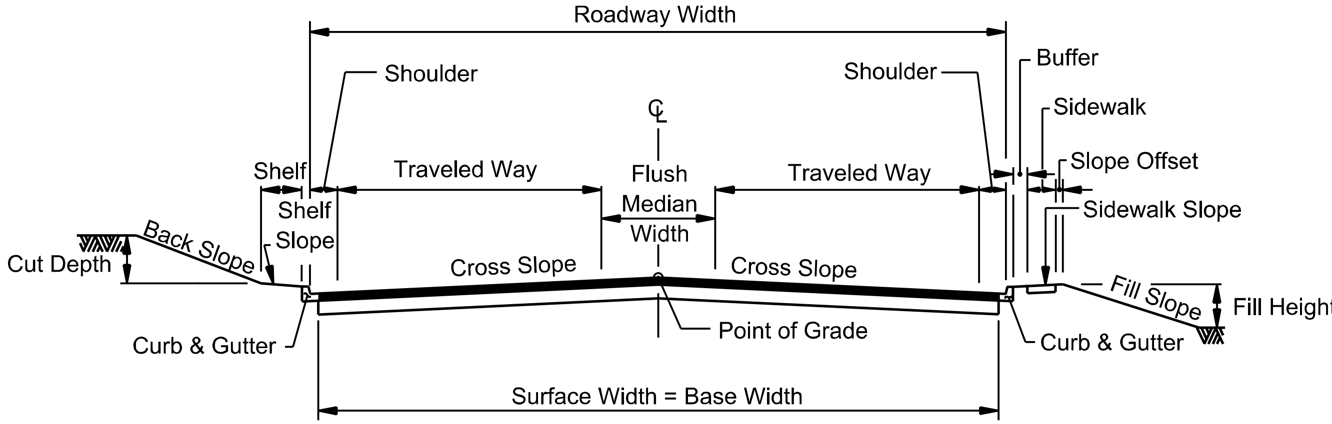
27. Vertical Curb. A longitudinal element, typically concrete, placed at the roadway edge for delineation, to control drainage, to control access, etc. Vertical curbs may range in height between 6 inches and 12 inches with a face no steeper than 1 horizontal (H) to 6 vertical (V).



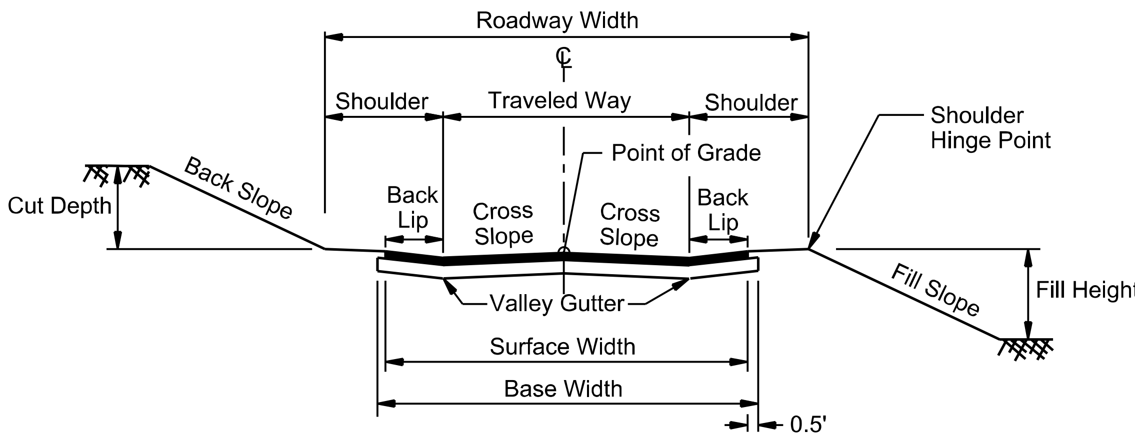
FREEWAY NOMENCLATURE
Figure 7.1-A



RURAL NOMENCLATURE
Figure 7.1-B



URBAN NOMENCLATURE
(Curb and Gutter)
Figure 7.1-C



URBAN NOMENCLATURE
(Valley Gutter)
Figure 7.1-D

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7.2 ROADWAY SECTION

7.2.1 Typical Cross Sections

Typical cross sections are graphical representations with dimensions showing the width, limits and slopes of the various cross sectional elements the designer uses for a particular project. Typical cross sections generally illustrate one side in a fill section and the other side in a cut section for both normal crown and superelevated sections. They generally detail the thickness, depth and layers of the pavement structural components. Typical cross sections for local roads and streets, collectors, arterials and freeways are presented in Chapters 14 through 17, respectively.

Chapters 14 through 18 provide the minimum criteria for lane widths, shoulder widths and other cross section elements.

7.2.2 Pavement Surface Type

For the selection of pavement type, the designer should consider the following factors:

- volume and composition of traffic,
- soil characteristics,
- weather,
- historical performance of various pavement types in the project area,
- energy conservation,
- availability of materials,
- costs, and
- life cycle costs.

Pavements are generally classified into one of three categories — high, intermediate and low. The selection of the appropriate pavement type for a particular road, street or highway is primarily based upon the anticipated volume and type of traffic. For more information on pavement selection, contact the Pavement Design Engineer. The Department uses the following pavement types:

1. High-Type Pavements. High-type pavements are classified as asphaltic concrete (flexible) or concrete (rigid). Use this pavement type wherever high volumes of traffic are expected. Adequate design and construction techniques should allow pavements to retain their cross sectional shape, smooth riding qualities, drain properly and maintain good skid-resistant properties throughout their expected service life. The primary objective in the selection, design and construction of high-type pavements is to ensure maximum performance. The proper design and construction avoids performing non-routine maintenance and the resultant interruption and annoyance to traffic operations; as well as, the associated increased cost to the responsible governmental entity and the highway user.

2. Intermediate-Type Pavements. Intermediate pavements are designed to lesser criteria than the high-type pavement. Intermediate-type pavements are a variety of bituminous surface treatments on a prepared base. These are generally used on collectors and local roads and streets.

3. Low-Type Pavements. Low-type surfaces are generally prevalent on very low volume roads and normally consist of earth-type base (sand-clay) and earth base with macadam, stabilized aggregate or some other type of stone surface. The Department has a very limited number of roadways with this type of pavement.

7.2.3 Traveled Way

The traveled way is the area upon which vehicles travel. The traveled way has a great influence on the operations and safety of a highway facility.

7.2.3.1 Lane Widths

Lane widths range from 9 to 14 feet, with the 12-foot width being the most common practice for State highway facilities. Lesser widths may be considered on 3R projects, see Chapter 18 "3R Projects (Non-Freeways)," and local roads and streets, see Chapter 14 "Local Roads and Streets." Consider the following factors when selecting lane widths:

1. Safety. Lane widths are one of many factors that may affect the roadway safety. The lane width of a roadway influences the comfort of driving, operational characteristics and, in some instances, the likelihood of crashes.

2. Highway Classification. Highway classification and type are major determining factors in the selection of lane width. Generally, highway classification is a function of expected traffic usage. Arterials and freeways will have wider widths while collectors and local streets and roads may have narrower widths. Chapters 14 through 17 provide lane widths for the various functional classifications.

3. Context. In rural areas, there is typically sufficient right of way available to provide wider lane widths. In urban areas, restrictive right of way may limit the travel lane widths.

4. Traffic Volume. The volume of traffic is also a factor in determining lane width. For example, a wider lane provides desirable clearances between vehicles traveling in opposite directions on two-lane, two-way rural highways when high traffic volumes and high percentages of commercial vehicles are expected.

5. Capacity. In general, wider lanes have a greater capacity than narrower lanes because motorists are less inhibited by the close proximity of adjacent traffic. This results in a higher running speed and, in some instances, increases capacity. However, widths greater than 12 feet do not necessarily increase traffic capacity. Therefore, the *Highway Capacity Manual* uses the 12-foot travel lane as the base width for determining capacity and reduction factors are provided for narrower lanes.

6. Lateral Obstructions. Consider the location of lanes with respect to curbs and other lateral obstructions. Motorists tend to avoid close obstacles; therefore, wider lanes are desired. The absence of a usable shoulder and the close proximity of obstructions to the edge of the traveled way also influence driver behavior. For more information on lateral obstructions and the effects on driver behavior, see the AASHTO *Roadside Design Guide* and the *Highway Capacity Manual*.

7. Trucks. Significant volumes of trucks influence the lane width selection. The size and location of trucks, within their respective travel lane, have a similar effect as a lateral obstruction on both opposing and adjacent traffic. Trucks tend to cause other traffic to travel at reduced running speeds, which reduces overall capacity.

7.2.3.2 Provisions for Bicycles on Traveled Way

Provisions for bicycles are an important consideration where new highways are being constructed or existing highways are being widened or otherwise improved. This is particularly true for areas that have adopted bicycle plans. In many instances, the following measures can considerably enhance highway safety and provide capacity for bicycle traffic:

- paved shoulders with a minimum usable shoulder width of 4 feet,
- travel lanes 14 feet wide (or wider) if there is limited shoulder width or curb and gutter,
- bicycle-safe drainage grates,
- smooth riding surfaces,
- separate facilities where bicycle traffic is expected to be high and where additional right of way can be obtained reasonably,
- at-grade manhole covers, and
- evaluation of rumble strips.

For additional guidance on provisions for bicyclist, see Section 11.8 of this *Manual*, Engineering Directive 22 “Considerations for Bicycle Facilities” and the AASHTO *Guide for the Development of Bicycle Facilities.*

7.2.3.3 Traveled Way Cross Slopes

The purpose of the pavement cross slope is to promote rapid removal of drainage from the pavement surface while enabling the driver to maintain control of the vehicle. Because low-type pavements are loose and pervious, they require a greater cross slope than high-type surfaces in order to reduce saturation of the unpaved surface and base materials. Figure 7.2-A presents cross slopes for the various types of pavements used by the Department.

Surface Class	Surface Type	Cross Slope
High	Hot Mix Asphalt Concrete Surface Course or Portland Cement Concrete on a Prepared Base.	2.00% (50H:1V) for first 2 lanes
Intermediate	Bituminous Surface Treatment on a Prepared Subbase.	2.00% (50H:1V)
Low	Earth Base/Stabilized Aggregate or Macadam.	2.50% (40H:1V) or 4.00% (25H:1V)

NORMAL PAVEMENT CROSS SLOPES
Figure 7.2-A

Cross slopes of 2.00 percent are permitted for up to two lanes plus one half the width of a flush median. Pavement beyond the second lane should have a cross slope of 2.50 percent. This is for travel lanes as well as auxiliary lanes. If a roadway section has curb and gutters and the profile grade is less than 2.00 percent, the designer may consider using a cross slope of 2.50 percent for the outside lane to improve drainage.

The following further describes the cross slopes used by the Department:

1. Two-Lane Highways. Crown the traveled way pavement at the centerline and use a cross slope as shown in Figure 7.2-A away from the centerline.

2. Four-Lane Divided Highways (Narrow Median). Crown the traveled way pavement at the inside edge of the traveled way and use a cross slope of 2.00 percent away from the median for all lanes.

3. Four-Lane Divided Highways (Wide Median). Crown the traveled way pavement at the centerline of each roadway and use a cross slope of 2.00 percent away from the centerline for all lanes.

4. Three-, Five- or Seven-Lane Highways (with TWLTL). Crown the pavement at the center of the TWLTL and use a cross slope of 2.00 percent away from the centerline for all lanes on three- and five-lane highways. For a seven-lane section, use a cross slope of 2.5 percent for the outside lanes.

5. Six-Lane Highways with a Concrete Median Barrier (CMB). The following will apply:

   a. CMB Raised. Crown the median at the centerline of the CMB and use a slope of 2.00 percent away from the center for the inside shoulders and for the first two travel lanes adjacent to the inside shoulder. Use a slope of 2.50 percent for the third lane breaking away from the outside edge of the second travel lane.

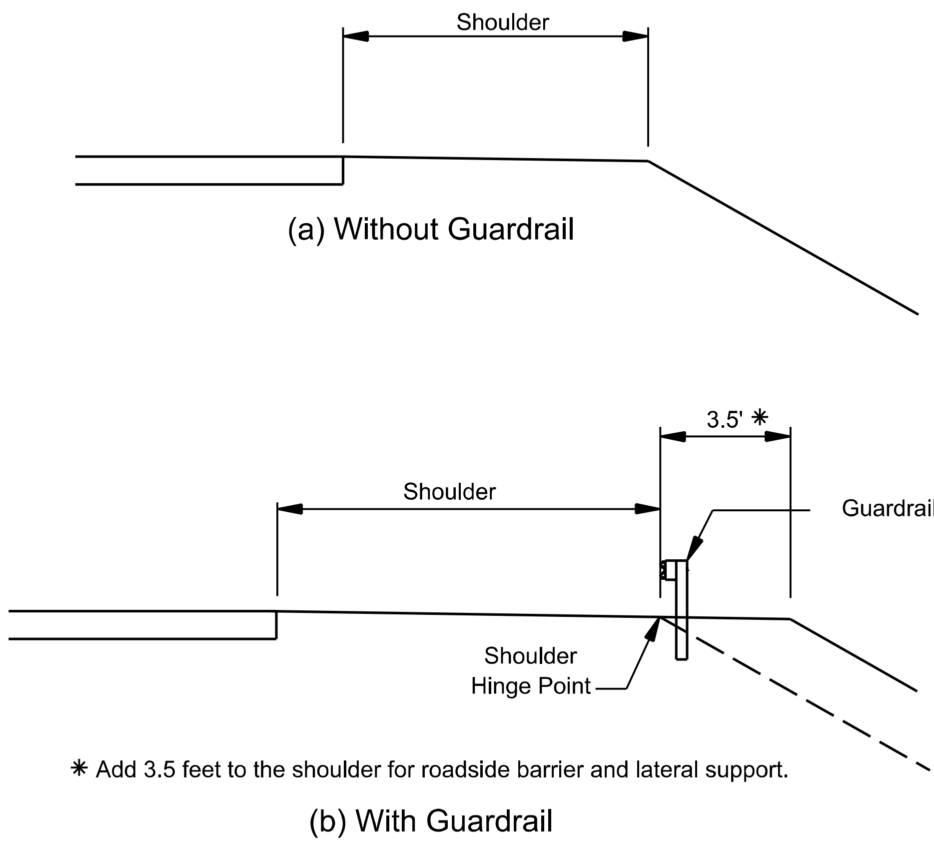
   b. CMB Depressed. When the median is lower than the adjacent traveled way, crown the traveled way between the first and second travel lanes (i.e., one lane sloping to the inside and two lanes sloping to the outside. The cross slope on all travel lanes will be 2.00 percent. The inside shoulder is sloped towards the CMB at a rate of 4.00 percent. When retrofitting existing facilities, do not break more than two lanes to the inside.

6. Bike Lanes. Where there is an auxiliary lane adjacent to the bike lane, the bike lane cross slope is the same as the through lane.

7.2.4 Shoulders

7.2.4.1 Function

Shoulders are defined as that portion of the roadway contiguous to the traveled way. They extend from the edge of the travel lane to the intersection of the foreslope. Shoulders may either be earth type or paved or a combination of both. Useable shoulders constitute the actual width of shoulder available for emergency stopping. Figure 7.2-B provides examples of shoulders.



SHOULDERS
Figure 7.2-B

Shoulders serve many functions, some of which include:

- providing structural support for the traveled way,
- increasing highway capacity,
- encouraging uniform travel speeds,
- providing space for emergency and discretionary stops,
- improving roadside safety by providing more recovery area for run-off-the-road vehicles,
- providing a sense of openness,
- improving sight distance around horizontal curves,
- enhancing highway aesthetics,
- facilitating maintenance operations,
- providing additional lateral clearance to roadside appurtenances,
- facilitating pavement drainage,
- providing space for pedestrian and bicycle use, and
- providing space for bus stops.

7.2.4.2 Shoulder Width

Shoulder widths will vary according to functional classification, traffic volumes, and urban or rural location. Chapters 14 through 18 present the shoulder width criteria for the various conditions.

In addition, guardrail can influence the shoulder width. Where guardrail is provided, increase the width of the shoulder by 3.5 feet to maintain the desirable useable width and to provide support for the guardrail. See Figure 7.2-B.

7.2.4.3 Shoulder Cross Slopes

Greater cross slopes are provided on shoulders than on adjacent travel lanes for two reasons: 1) the runoff carried across the shoulder is a combined total of both the adjacent travel lane and the shoulder; and 2) in many cases, the shoulder surface material is rougher than the adjacent travel lane requiring a steeper slope to maintain a similar flow rate. Not all shoulders are paved, so it is necessary to remove the water as rapidly as possible before it penetrates the shoulder with the potential of reducing its structural support capabilities. Normal shoulder slopes are shown in Figure 7.2-C. See Section 5.3.5 for the treatment of shoulders through superelevated curves for new construction and reconstruction projects. On 3R projects, existing shoulder slopes may be retained; see Chapter 18 “3R Projects (Non-Freeways).”

All shoulders should be structurally adequate to support truck usage for emergency purposes. In addition, ensure shoulder materials are sufficiently stable to provide lateral support of the adjacent pavements.

For maintenance operations, it is advantageous for the shoulders to be delineated from the through traffic lanes. This generally can be accomplished by using a different surface treatment, a different surface gradation and finish, pavement markings if the same surface material is used for both the traveled way and shoulder, and rumble strips.

Surface Class	Surface Type	Cross Slope
Paved	Hot Mix Asphalt Concrete or Portland Cement Concrete	4.00%* (25H:1V)
Turf	Compacted Earth with Grass Surface	8.00% (12.5H:1V)

* *If the paved shoulder is 4 feet or less, use the adjacent travel lane cross slope.*

NORMAL SHOULDER CROSS SLOPES
Figure 7.2-C

7.2.5 <u>Rumble Strips</u>

Rumble strips are a proven, cost-effective way to reduce crashes. They alert drivers of lane departures by providing an audible and vibratory warning. Shoulder rumble strips help reduce roadway departures; whereas, centerline rumble strips reduce vehicular crossovers on undivided highways.

Engineering Directive 53 “Installation of Rumble Strips” provides the warrants for the placement of rumble strips on freeways and other highways. The *SCDOT Standard Drawings* provide the design details for milled rumble strips and stripes, and their application on freeways and other highways.

Place a note on the plan sheet showing “Begin Mill-in Rumble Strip” and “End Mill-in Rumble Strip” with an arrow to the appropriate location; see the *SCDOT Standard Drawings*.

7.2.6 Auxiliary Lanes

7.2.6.1 General Guidance

Auxiliary lanes are any lanes beyond the basic through travel lanes. They are intended for use by vehicular traffic for specific functions. The following will apply to the design of auxiliary lanes:

1. Width. The width of an auxiliary lane is typically the same as that of the adjacent through lane. Auxiliary lane widths for various classifications of highways are provided in the design tables in Chapters 14 through 17.

2. Types. Auxiliary lanes include:

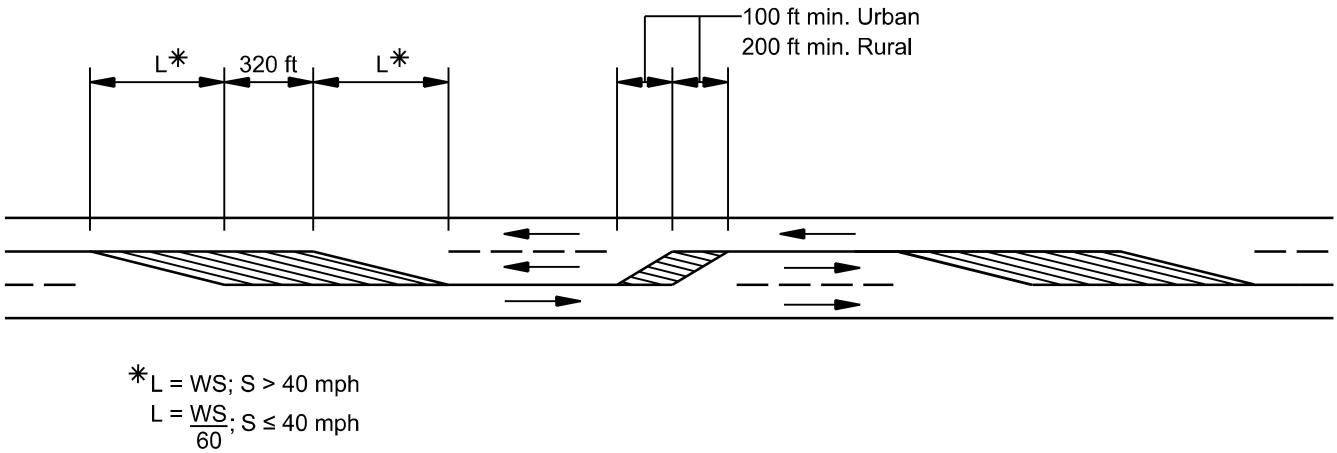
    - single left- and right-turn lanes at intersections,
    - double left- and right-turn lanes at intersections,
    - truck-climbing lanes,
    - acceleration/deceleration lanes at interchanges or intersections,
    - weaving lanes within an interchange,
    - continuous auxiliary lanes between two closely spaced interchanges,
    - parking lanes, and
    - passing lanes.

3. Shoulders. The shoulder width adjacent to the auxiliary lane should be the same as the normal shoulder width for the approaching roadway. At a minimum, the width should be 6 feet assuming the roadway has a shoulder width equal to or greater than 6 feet.

4. Cross Slope. The cross slope for an auxiliary lane will depend on the number of lanes and cross slope of the adjacent traveled way. If the auxiliary lane is the second lane from the crown, provide a cross slope of 2.00 percent. If the auxiliary lane is the third or fourth lane from the crown, use a cross slope of 2.50 percent. See Section 7.2.3.3 for additional information on cross slopes.

7.2.6.2 2+1 Roadways

The 2+1 roadway is a continuous three-lane cross section with striping to provide for passing in alternate directions; see Figure 7.2-D. A 2+1 roadway may be considered as an alternative to two-lane highways where passing lanes are necessary to obtain the desired level of service, but the traffic volumes are not high enough to justify a four-lane facility. The decision to use the 2+1 roadway will be determined on a case-by-case basis based on long-range planning objectives for the facility, available right of way, existing cross section, topography and the need to reduce platooning and passing problems.



2+1 ROADWAY
Figure 7.2-D

When designing 2+1 roadways, consider the following:

1. Sight Distance. Provide stopping sight distance throughout the 2+1 roadway. Desirably, provide decision sight distance to the lane drops and at intersections.

2. Level of Service. A 2+1 roadway will typically operate at least two levels of service higher than a conventional two-lane highway serving the same traffic volume.

3. Capacity. Do not consider a 2+1 roadway where current or projected traffic volumes exceed 1200 vehicles per hour in one direction of travel. A four-lane facility is generally more efficient at these traffic volumes.

4. Terrain. Only use 2+1 roadways in level or rolling terrain. In mountainous terrain or on isolated steep grades, a climbing lane is generally more appropriate.

5. Cross Sections. The lane and shoulder widths should match the adjacent section of the two-lane highway.

6. Major Intersections. Locate major intersections in the transition area between opposing passing lanes, and provide striping for left-turn lanes at the intersection, as applicable. Low-volume intersections and most driveways may be accommodated within the passing lane sections.

7. Signing and Pavement Markings. Signing and pavement markings for a passing lane should meet the criteria in the *MUTCD*.

7.2.7 Parking

7.2.7.1 Guidelines

Adjacent land use may create a demand for on-street parking along an urban street. Parking lanes provide convenient access for motorists to businesses and residences. However, on-street parking reduces capacity, impedes traffic flow and increases crash potential.

The decision to retain existing on-street parking or to introduce on-street parking will be based on a case-by-case assessment in cooperation with the local community. Evaluate the following factors:

- prior crash experience or potential safety concerns;
- impacts on the capacity of the facility;
- current or predicted demand for parking;
- actual needs versus existing number of spaces;
- alternative parking options (e.g., off-street parking);
- input from local businesses;
- impacts on right of way;
- impacts on bicyclists and pedestrians;
- pedestrian accessibility;
- construction costs; and
- projected traffic volumes.

7.2.7.2 Parking Types

The two basic types of on-street parking are parallel and angle parking. Parallel parking is the preferred arrangement where street space is limited and traffic capacity is a major factor. Angle parking provides more spaces per linear foot than parallel parking, but a greater cross street width is necessary for its design. The total entrance and exit time for parallel parking exceeds that required for angle parking. Parallel parking also requires a vehicle to stop in the travel lane and await an opportunity to back into the parking space. However, angle parking requires the vehicle to back into the lane of travel when adjacent parked vehicles may restrict sight distance and where this maneuver may surprise an approaching motorist.

7.2.7.3 Design

In many communities, local codes or regulations dictate the dimensions for parking layouts. For guidance on parking stall widths, stall layouts and other parking design criteria, the designer should review the Institute of Transportation Engineers' *Traffic Engineering Handbook*.

7.2.7.4 Location

For most sites, conduct a parking occupancy turnover study and a sight distance evaluation. In addition to State and local regulations when locating parking spaces, consider the following:

- Prohibit parking within 20 feet of any crosswalk.

- Prohibit parking at least 10 feet from the beginning of the curb radius at mid-block approaches (e.g., alleys, driveways).

- Prohibit parking within 50 feet of the nearest rail of a railroad/highway crossing.

- Prohibit parking from areas designated by local traffic and enforcement regulations (e.g., near school zones, fire hydrants). See local ordinances for additional information on parking restrictions.

- Prohibit parking near bus stops (see Section 11.8).

- Prohibit parking within 30 feet on the approach leg to any intersection with a flashing beacon, stop sign or traffic signal.

- Prohibit parking on bridges or within a highway tunnel.

- Eliminate parking across from a T- intersection.

- Prohibit parking in the intersection sight triangle.

7.2.8 Curbs

7.2.8.1 Usage

Curbs are often used on urban facilities to control drainage, delineate the pavement edge, channelize vehicular movements, control access, limit right of way needs, provide separation between vehicles and pedestrians and present an attractive appearance. Curbs are generally not used in rural areas. For urban and suburban areas, determining if curbs will be used depends upon many variables, and the decision will be made on a case-by-case basis. Evaluate the following factors to determine whether a curbed section is preferred:

- local preference,
- drainage impacts,
- construction costs,
- impacts on maintenance operations,
- roadside safety impacts,
- sidewalks (see Section 7.3.3),
- control of access to abutting properties,
- impacts on traffic operations,
- right-of-way restrictions,
- vehicular speeds, and
- potential for future widening.

Curbs used along the outside pavement edges serve a variety of functions (e.g., drainage control, delineating edges of pavement and pedestrian walkways, aesthetics, reduce right of way). Curbs are also used as aids in channelizing vehicle movement at intersections, controlling access points and providing lateral support of the roadway or shoulder pavement. The use of curbs predominates in urban areas as opposed to rural.

Curbs may be provided on urban ramps if they are located at outside edge of the paved shoulder. If curbing is provided on ramp, it should be within the gore area or along the ramp proper. Curbing should not be provided along the freeway mainline.

7.2.8.2 Types

There are two basic types of curbs, vertical curb and sloping curb. Either curb type may be constructed with an integral gutter to form a curb and gutter section. Curbs are constructed of concrete either cast-in-place or extruded. Where curb and gutter is used, whether vertical or sloping curb, the gutter portion is not considered a part of the traveled way.

The design details for the following curb types used by the Department are provided in the *SCDOT Standard Drawings*:

1. Vertical. Vertical curbs are normally 6 inches in height with steep vertical faces (1H:6V). They are intended to discourage vehicles from leaving the traveled way. Do not use vertical curbs adjacent to travel lanes where design speeds exceed 45 miles per hour. Instead, use a sloping curb placed at the outer edge of the shoulder.

2. Sloping. Sloping curbs are generally 6 inches in height with a sloped face (4H:5V). Sloping curbs will allow a vehicle to mount the curb and can be used with any design speed. If the design speed is greater than 45 miles per hour, place the curb on the outside edge of the shoulder. For freeways and ramps, only use sloping curbs at the outside edge of the shoulder, where necessary. With this curb type, the designer needs to select the appropriate drainage structure.

3. OGEE. The OGEE curb is a gently rounded curb approximately 4 inches high. Vehicles can easily transverse over the curb. It can be used for roadside access (e.g., driveways) where curb cuts are not desired. Where drainage structures are required, provide a special detail to transition the OGEE curb to the standard drainage inlet. Accessibility and on-street parking should be evaluated due to the ease of mounting OGEE curb.

4. 9 inch x 15 inch. The 9 inch x 15 inch curb can be used with raised medians where the adjacent pavement is sloped away from the median. It is a sloping curb that can be easily mounted.

SCDOT also defines the curb based on the drainage direction. Catch curbs drain toward the curb and gutter. Spill curbs drain away from the curb and gutter. SCDOT curb types are typically catch curbs, but also can be a spill curb.

7.2.8.3 Accessibility

Provide curbs ramps at all pedestrian crossings to provide accessibility. See the *SCDOT ADA Transition Plan* and *SCDOT Standard Drawings* for details on the design and location of curb ramps.

7.2.9 Valley Gutter

Valley gutters are commonly used on local roads and streets. See Figure 7.1-D for a cross section with a valley gutter. On State routes, a valley gutter may be used to transition from a shoulder section to a curb and gutter section. The length of the transition from the shoulder section to the curb and gutter section should be based on the following:

- $L = WS$, where $S > 40$ miles per hour, feet; or
- $L = WS^2/60$, where $S \leq 40$ miles per hour, feet.

W is the difference in width between the shoulder width and the curb and gutter width.

7.3 ROADSIDE ELEMENTS

7.3.1 Roadside Safety

See the AASHTO *Roadside Design Guide* for specific criteria for clear zones based on the foreslope, ditch width and backslope combinations; barrier warrants; roadside and median barriers; barrier layouts; and impact attenuators.

7.3.2 Slopes and Ditches

7.3.2.1 Purpose

Earth slopes are required to provide safe roadside and median ditches adjacent to highway facilities and to provide a stable transition from the highway profile to adjacent terrain features. Flat slopes also facilitate turf establishment and are often required for soil stability. Flat and well-rounded side slopes, combined with proper roadway elevations above natural ground lines also enhance the roadway aesthetically as well as provide easy accessibility with regard to maintenance operations.

Using broad flat slopes on roadside ditches, which are visible to the driver, lessen the feeling of restriction and add considerably to a driver’s willingness to use the shoulder and earth slope area in emergencies.

The designer should evaluate slopes in accordance with the *SCDOT Geotechnical Design Manual*. Coordinate slope modifications with the geotechnical designer.

7.3.2.2 Fill Slopes

Fill slopes are the foreslopes extending outward and downward from the shoulder hinge point to intersect the natural ground line. The slope criteria is dependent upon the functional classification, fill height, urban/rural location and the presence of curbs. Although Chapters 14 through 17 provide design guidance for fill sections for each of the classifications of roadways, the designer must also consider geotechnical design requirements, right-of-way restrictions, utility considerations, roadside safety and roadside development in determining the appropriate fill slope. Figure 7.3-A provides the fill slope ratios for typical conditions measured from the shoulder hinge point. The designer should evaluate each location to determine the appropriate fill slope ratios; however, 4:1 or flatter slopes are encouraged because they are easier to mow or otherwise maintain and safer to negotiate for errant vehicles.

Fill Height	*Foreslope Ratio
≤ 5 feet	6H:1V
5 < 10 feet	4H:1V
≥ 10 feet	2H:1V

******The designer should provide a transition between different foreslope ratios; see Section 7.3.2.4*.

TYPICAL FILL SLOPE RATIOS
Figure 7.3-A

7.3.2.3 Cut Slopes

The following applies to cut slopes:

1. Earth Cuts. In earth cuts on facilities without curbs, roadside ditches are provided to control drainage. The ditch section includes the foreslope, ditch width (typically a V ditch is used) and backslope as appropriate for the facility type. On facilities with curbs and no sidewalks, a shelf (typically 6 feet measured from the back of curb) is provided, and the backslope of 2H:1V is located beyond the shelf. See the typical cross sections in Chapters 14 through 17.

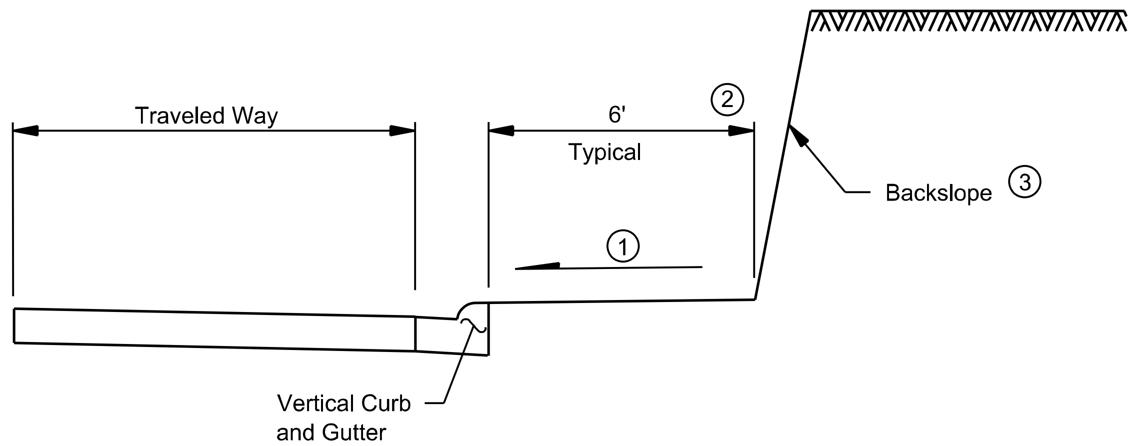
2. Rock Cuts. In rock cuts, the backslope generally is steeper than earth cuts; see Figure 7.3-B. For large cuts, benching of the backslope may be required. The geotechnical designer is responsible for determining the appropriate rock cut slopes.

   The designer should perform a hydraulic analysis to evaluate the need to control cascading water from the top of the cut and to determine the conveyance needs or roadside drainage at the toe of the cut.

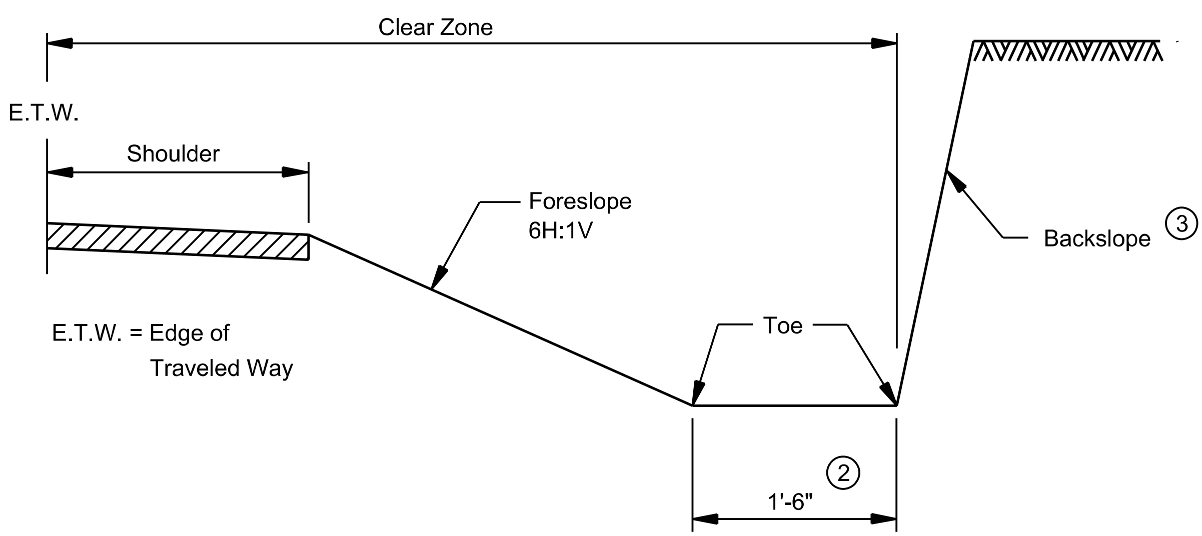
3. Daylighting. Daylighting extends or flattens backslopes and can provide several benefits, including:

   - enhancing aesthetics,
   - enhancing roadside safety,
   - providing needed fill material,
   - removing undesirable features,
   - obliterating existing roadbeds, and
   - providing convenient outfall points for roadside drainage.

   The decision to use daylighting should be made on a case-by-case basis in coordination with the Project Development Team.



(a) Curb and Gutter Section



(b) Ditch Section

Notes:

① *Use 50H:1V if sidewalks are presented or anticipated. Use 30H:1V if sidewalks are not present or anticipated.*
② *Discuss with the geotechnical designer to determine extra width needed for falling rock.*
③ *Backslopes in rock may require benching. Contact the geotechnical designer for guidance.*

TYPICAL ROCK CUT SECTIONS
Figure 7.3-B

7.3.2.4 Slope Transitions

The designer should flatten and round side slopes to fit the topography consistent with the site conditions, roadside safety and cost effectiveness of the design. Gradual transitions from cut to fill slopes, or within a cut or fill slope, will avoid unattractive bulges and sharp depressions. Desirably, provide a 100-foot transition for every 1:1 slope difference (e.g., provide a 300-foot transition between a 6:1 slope and a 3:1 slope).

7.3.2.5 Ditch Section

A properly designed roadside ditch will ensure the proper drainage of the pavement subgrade and the adequate conveyance of surface flow without creating erosion. Roadside ditches are provided adjacent to embankment locations and in cut sections.

The Department typically uses a V-ditch section along facilities without curb and gutter. The ditch section includes the foreslope and backslope. If the longitudinal gradient is greater than 3 percent, the hydraulic designer will review the ditch design as well as include specially designed sediment control items and lining, as appropriate.

7.3.3 Sidewalks

The designer should evaluate the need for pedestrian accommodations on every project.

In determining the sidewalk design, the designer should consider the following:

1. Width. The minimum sidewalk width is 5 feet. However, consider wider sidewalks along streets with schools or where on-street parking and bus stop areas are located. In commercial areas, wider sidewalks may allow for the increased volume of pedestrian traffic normally associated with these areas. Sidewalk usage and widths are determined using the *Highway Capacity Manual*.

   The designer should also evaluate the width considering compatibility with local city and community criteria.

2. Placement. Sidewalks should be placed as far as practical from the travelled way, preferably at the right-of-way line; however, no closer than 0.5 foot from the right-of-way line. The designer should consider the following:

   a. Curb and Gutter. Desirably, provide a 3-foot buffer area between the curb and the sidewalk; see Item #5. If there is insufficient right of way to provide the 0.5-foot space to the right-of-way line, the sidewalk may be placed adjacent to the curb and gutter.

   b. Valley Gutter. A valley gutter section with sidewalk is discouraged in high pedestrian traffic areas. The Project Development Team should discuss pedestrian options during the design field review if a valley gutter with sidewalk is recommended. Desirably, include a buffer strip between the valley gutter and the sidewalk. If there is insufficient right of way to provide the 0.5-foot space to the right-of-way line, the sidewalk may be placed adjacent to the outside edge of the valley gutter.

c. Shoulder. For all new/reconstruction projects, place the sidewalk beyond the ditch, but no closer than 0.5 foot from the right-of-way line. For retrofit projects (e.g., Safe Routes to School), the sidewalk may be placed within the shoulder and as far as practical away from the traveled way. Only consider this option where there are no other practical options available. Include at least a 1-foot shelf between the sidewalk and edge of the ditch section.

3. Appurtenances. The designer should also consider the impacts of roadside appurtenances within the sidewalk (e.g., fire hydrants, parking meters, utility poles, signs). These elements will reduce the effective width because they interfere with pedestrian activity. Preferably, place these appurtenances behind the sidewalk. If they are placed within the sidewalk, the sidewalk should have a minimum clear width of 4 feet; desirably, a 5-foot clear width. Measure the clear width from the edge of the appurtenance to the edge of the sidewalk. A 4-foot minimum is necessary to meet the accessibility requirements; see the *SCDOT ADA Transition Plan*.

4. Cross Slope. The maximum cross slope on the sidewalk is 2 percent sloped towards the roadway.

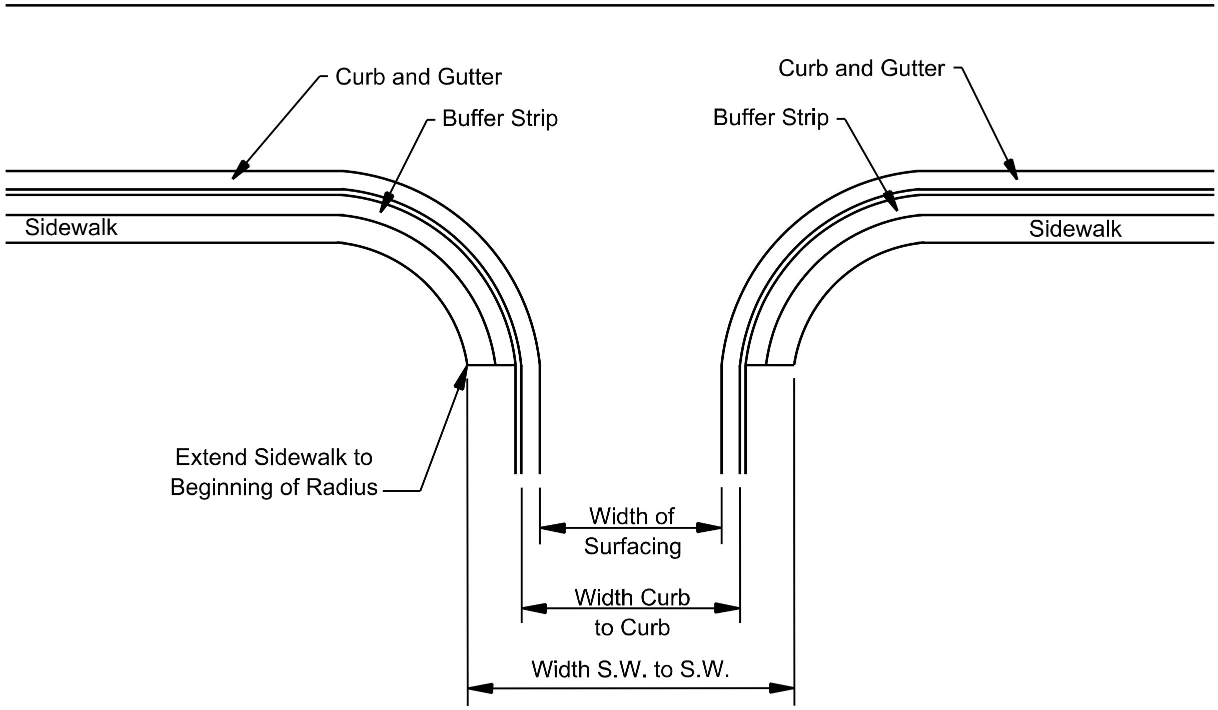
5. Buffer Areas. If the available right of way is sufficient, a grass buffer area between the curb and sidewalk is desirable. The buffer area should desirably be 3 feet wide; however, lesser widths may be considered in constrained urban environments. The buffer provides enhanced sight visibility at driveways/intersections, space for street and highway hardware (e.g., signs, hydrants, mailboxes) and groundcovers, and provides a greater distance between pedestrians and moving traffic or the opening of doors of parked cars.

6. Intersections. Extend sidewalks to the beginning of the corner radius on the adjacent roadway as shown in Figure 7.3-C. Note the extension of the sidewalk on Figure 7.3-C(b) beyond the corner radius in order to match the sidewalk on the opposite side. Discuss the location of sidewalk termini during the Design Field Review.

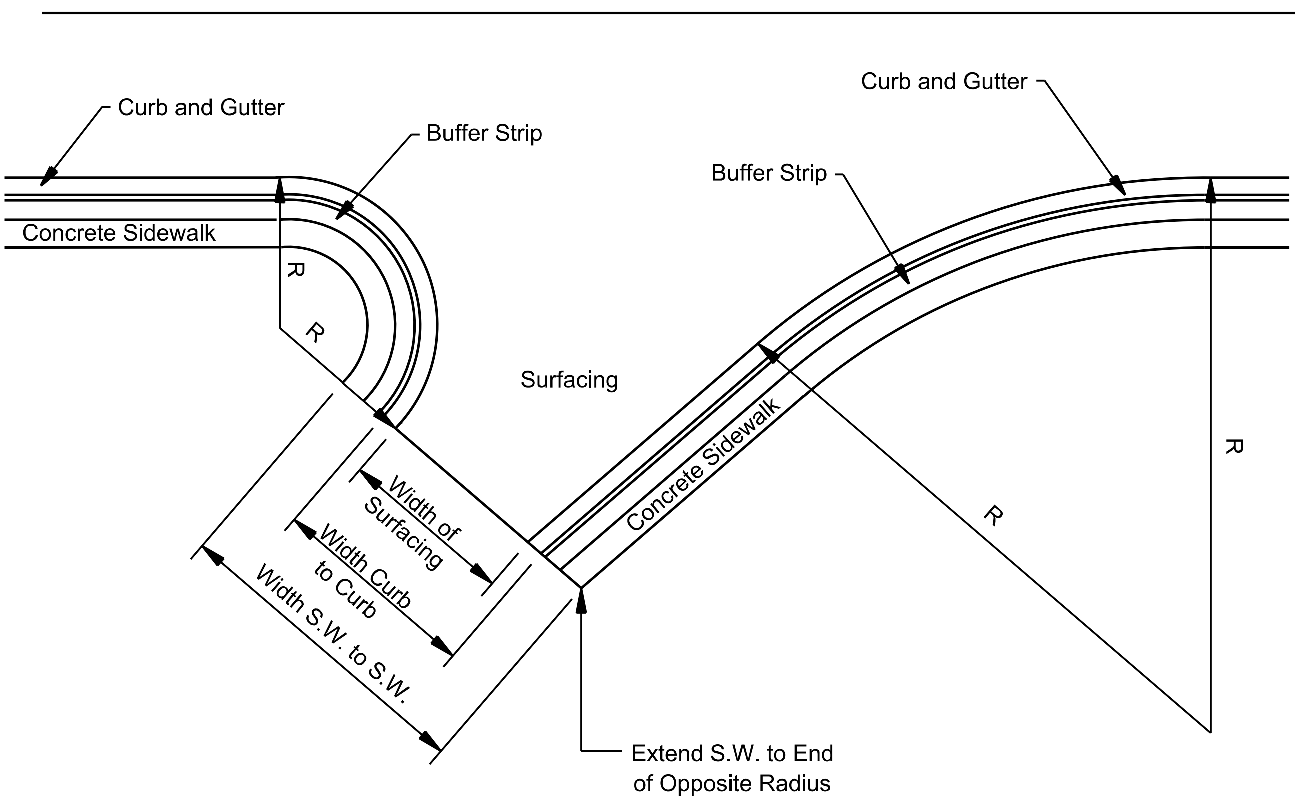
7. Pedestrian Crossings. Pedestrian crossings are normally located at intersections. In areas where there are curbs, provide curb ramps at crossing locations. See the *SCDOT Standard Drawings* and *SCDOT ADA Transition Plan* for additional guidance.

8. Pavement Material. Sidewalks are generally constructed of concrete and are 4 inches thick except at driveways where the thickness is increased to 6 inches.

9. Bridges. The structural designer is responsible for the dimensioning and structural design of all sidewalks on bridges.



(a) Perpendicular Intersection



(b) Angled Intersection

SIDEWALK EXTENSIONS AT INTERSECTIONS
Figure 7.3-C

7.3.4 Aesthetics

There are various methods in which to improve the visual impact of a roadway. Varying the cross section elements will typically improve the aesthetics of the roadway. This may include:

- increasing or decreasing the side slopes to reduce the magnitude of exposed cut and fill slopes;
- reducing ditch widths or depths to reduce the amount of cut, with the approval of the hydraulic designer;
- using slope rounding to blend cuts and fills into the natural ground;
- warping side slopes to match the natural landscape;
- retaining existing vegetation;
- using a raised or depressed median with plantings; and/or
- providing structures that match the natural landscape.

SPACER PAGE

7.4 MEDIANS

7.4.1 Functions

A median is defined as the area of a divided highway separating the traffic into opposing directions. The principal functions of a median are to:

- provide separation from opposing traffic,
- prevent undesirable turning movements,
- provide an area for deceleration and storage of left-turning vehicles,
- provide an area for storage of vehicles for emergency stopping,
- facilitate drainage collection,
- provide a recovery area for run-off-the-road vehicles,
- provide an area for pedestrian refuge,
- provide width for future lanes, and
- minimize headlight glare.

7.4.2 Median Types

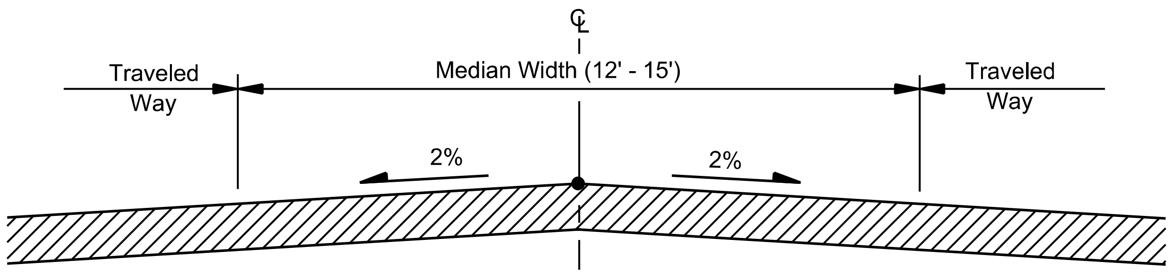
The decision on the median type to be used should be made as early as possible. Figure 7.4-A provides typical sections for the flush, flush with a concrete median barrier (CMB), raised and depressed medians.

7.4.2.1 Median Selection

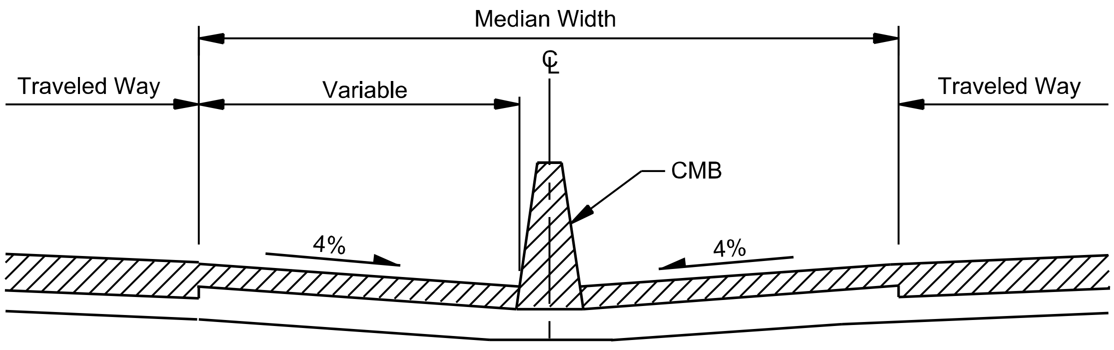
When selecting a median type, recognition must be given to urban/rural location, access needs, design speeds, availability of right of way, safety, crash history, capacity, intersection spacing, traffic signals, sight distance, turn-lane length, economics, environmental impacts, public appearance and functional classification. Higher functional classifications will warrant a greater effort in managing access to a street or highway and in retaining mobility. See Section 3.4.1 for a discussion on mobility.

On certain projects, more than one median type may be necessary and/or desirable. The length of a project will be a major influence in this determination. On relatively short highway sections, the number of different median types should be limited to a select few.

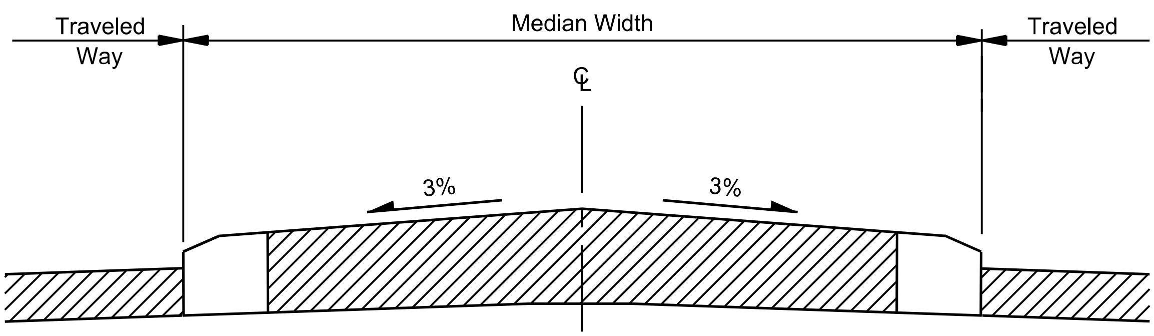
Non-traversable medians generally have a lower crash rate than flush medians. However, non-traversable medians will eliminate left-turn movements at some intersections and driveways, but may increase U-turn volumes at other locations on the same road or may divert some traffic to other roads.



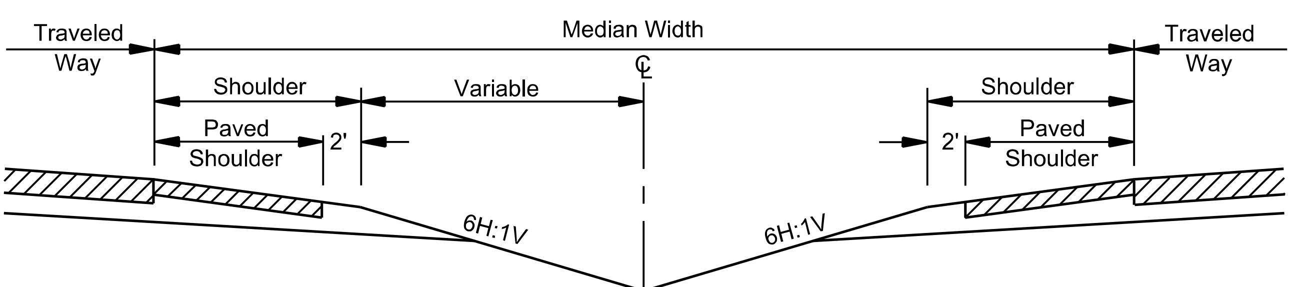
(a) Flush Median
Painted Left-Turn Lanes Or TWLTL



(b) Flush Median
With Concrete Median Barrier (CMB)



(c) Raised Median



(d) Depressed Median

MEDIAN TYPES
Figure 7.4-A

7.4.2.2 Flush Medians

A flush median is defined as the paved median surface at essentially the same plane as the adjoining traveled way. Flush medians are used most often on urban highways and streets where design speeds are 45 miles per hour or less. The following will apply:

1. Flush. Typical widths for a flush median can range from 4 to 30 feet. While medians 4 to 6 feet may be provided under restrained conditions, medians 12 to 30 feet wide provide a protected storage area for left-turning vehicles at intersections. To provide proper drainage, flush medians are typically crowned in the center with a cross slope of 2 percent in each direction.

2. Flush Median with TWLTL. Two-way, left-turn lanes (TWLTL) are considered flush medians. See Section 7.4.3 for guidance on TWLTL.

3. Flush with CMB. A flush median with a CMB may be used on urban freeways where the right of way may prohibit widening to the outside. Desirably, the medians should be sufficiently wide to allow for the addition of 12-foot wide travel lanes and 10-foot shoulders adjoining the CMB where there are three or more travel lanes in each direction. However, the predominant median width for most urban freeways in South Carolina is 36 feet. This allows for the use of two 12-foot travel lanes, a 2.5-foot wide CMB and two 4.75-foot wide shoulders.

7.4.2.3 Raised Medians

A raised median may be proposed for managing access, improving operations and improving safety. When used, the designer should consider the following:

1. Design Speed. Due to the curb associated with raised medians, they are generally only used where the design speed is 45 miles per hour or less. Higher design speeds may be considered, but normal shoulder widths will apply.

2. Width. Enough right of way should be available for widths that will provide space for the initial and future installation of left-turn lanes at public street intersections and high-traffic generator locations. At intersections, the raised portion is typically 4 feet wide. The length may be determined by need for control of turning maneuvers near the intersection.

3. Access Control. Raised medians restrict access to driveways and other private developments unless a median opening is provided.

4. Planted Medians. A raised median with plantings in the center may be proposed for aesthetic purposes. Use curbing to delineate green areas in the center of roadways. Ensure adequate drainage is provided. Give special attention to eliminate areas that trap water in transition from normal to superelevated sections. Cross slopes in the median are typically 30H:1V. Consider roadside safety concerns and sight distances when determining plantings in the raised median.

7.4.2.4 Depressed Medians

A depressed median is typically used on freeways and other divided rural arterials. For non-freeways, a depressed median is usually considered where managed access to the street and control of left-turn movements are desired. Depressed medians typically have good drainage characteristics and, therefore, are preferred on major highways.

Depressed medians should be as wide as practical to allow for the addition of future travel lanes on the inside while maintaining a sufficient future median width. The minimum width is 48 feet. This allows for the initial development of a depressed median with 6H:1V side slopes and a ditch with sufficient depth to accommodate the runoff. Avoid slopes steeper than 6H:1V. The 48-foot width allows for two future travel lanes with two 10-foot shoulders and a 4-foot CMB section. The maximum width for a depressed median is approximately 84 feet with 8H:1V side slopes. Beyond this, the two roadways of the divided facility are typically placed on independent alignments.

7.4.3 Two-Way Left Turn Lanes (TWLTL)

The applicability of a TWLTL is a function of the traffic conditions that result from the adjacent land use. Selection of a TWLTL should be coordinated with the local land use plan.

The designer should consider the following guidance:

1. Advantages. TWLTL offer several advantages when compared to no median; for example, TWLTL:

   - reduce travel time;
   - improve capacity;
   - reduce crash frequency, particularly of the rear-end type;
   - increase flexibility (e.g., the lane can be used as a travel lane during closure of a through lane); and
   - are generally preferred by drivers and owners of abutting properties.

2. Access Control. TWLTL may be inappropriate at some locations. TWLTL tend to increase rather than control access opportunities. Where better access control is desired (e.g., along arterials), the Project Development Team may need to consider replacing an existing TWLTL with a non-traversable median.

3. Locations. Consider providing a TWLTL:

   - in areas with a high number of existing driveways per mile (e.g., 10 to 35 driveways per mile on both sides of the street);
   - in areas of existing high-density commercial development;
   - in areas with substantial mid-block left turns; and/or

- on facilities with four travel lanes or less.

Do not provide a TWLTL if there is less than 200 feet of space available.

4. Speed. The use of TWLTL is not normally provided where the design speed exceeds 45 miles per hour. Their applicability to rural highways is typically near suburban areas or for roads passing through small towns.

5. Median Width. SCDOT generally requires a 15-foot TWLTL. Minimum TWLTL widths are provided in Chapters 14 through 18. To obtain the TWLTL width, the designer may consider the following:

    - reducing the width of existing through lanes and analyzing side road radius returns,
    - eliminating existing parking lanes and reconstructing curb and gutter and sidewalks,
    - reconstructing existing shoulders and ditches,
    - replacing existing shoulders and ditches with curb and gutter,
    - eliminating existing buffer areas behind curbs and reconstructing curb and gutter and sidewalks, and/or
    - acquiring additional right of way to expand the pavement width by the amount needed for the TWLTL.

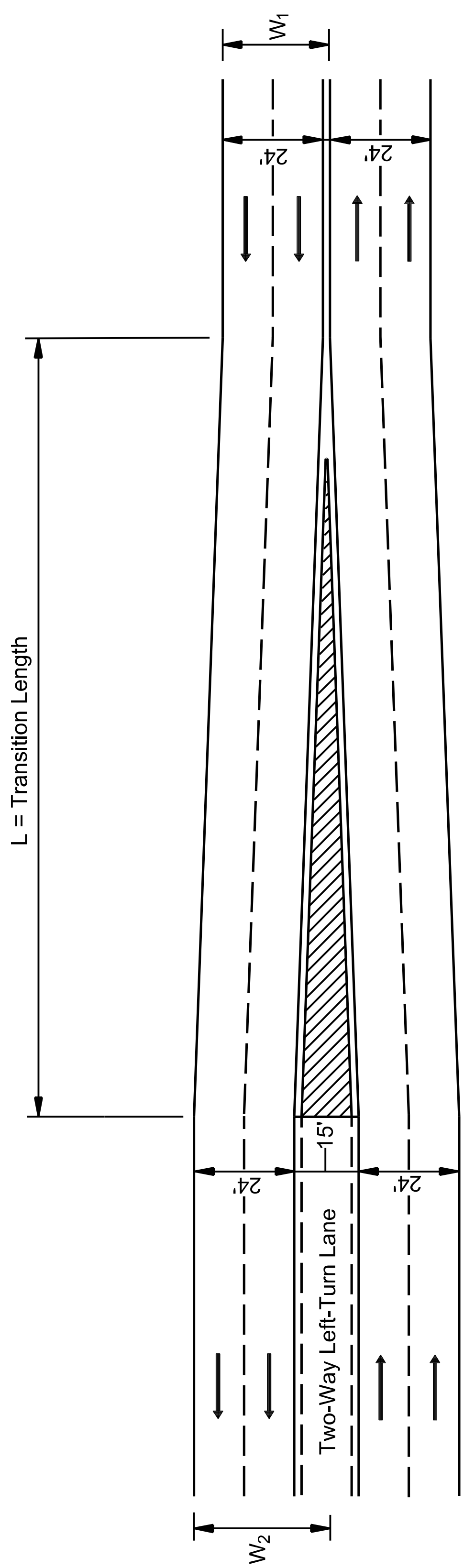
    The TWLTL width may be reduced to 12 feet to accommodate bicycle facilities on an existing roadway or 10 feet on a 3R project. Median widths less than 12 feet are not recommended where posted speeds are greater than 35 mph and the percentage of trucks, buses and recreational vehicles is greater than 5 percent of the AADT.

6. Intersection Treatment. At intersections with public roads, the designer should coordinate with Traffic Engineering to determine the appropriate design treatment of the TWLTL (i.e., end the TWLTL prior to the intersection or to continue the TWLTL through the intersection).

7. Operational/Safety Factors. Provide proper signing and stopping sight distance at the beginning and end of each TWLTL. Where a number of turning movements are expected into and out of entrances located close to a major intersection, it is desirable to provide a channelized design for the exclusive left-turn lane; see Section 9.5.

8. Transitions. Figure 7.4-B provides an example lane transition design for a four-lane roadway to a five-lane TWLTL facility. Figure 7.4-C provides an example lane transition design from a two-lane roadway to a five-lane TWLTL.

For additional information on the use and design of a TWLTL, see the AASHTO *A Policy on Geometric Design of Highways and Streets*.

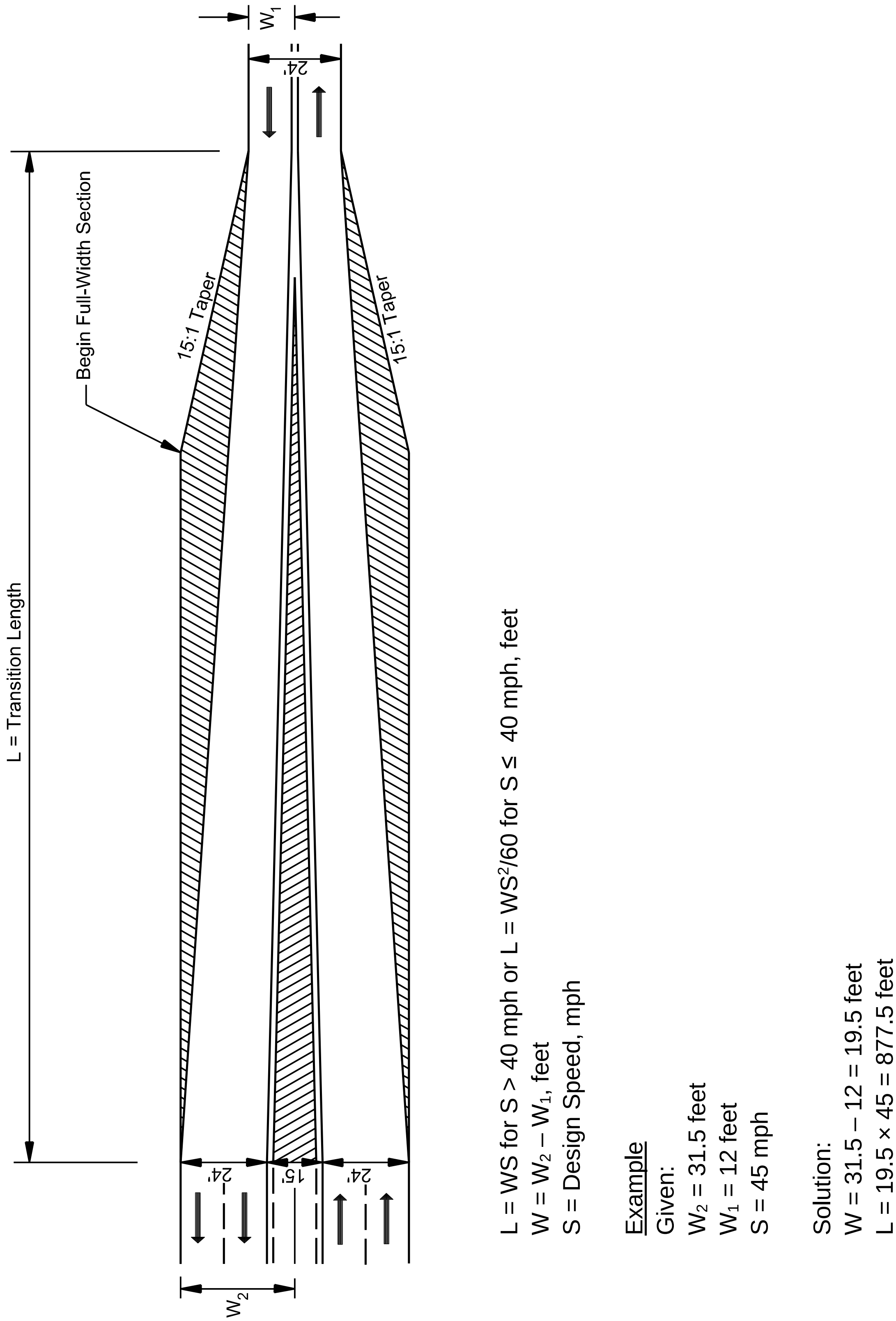


$L = WS$ for $S > 40$ mph or $L = WS^2/60$ for $S \leq 40$ mph, feet
$W = W_2 - W_1$, feet
S = Design Speed, mph

<u>Example</u>
Given:
$W_2 = 31.5$ feet
$W_1 = 24$ feet
$S = 40$ mph

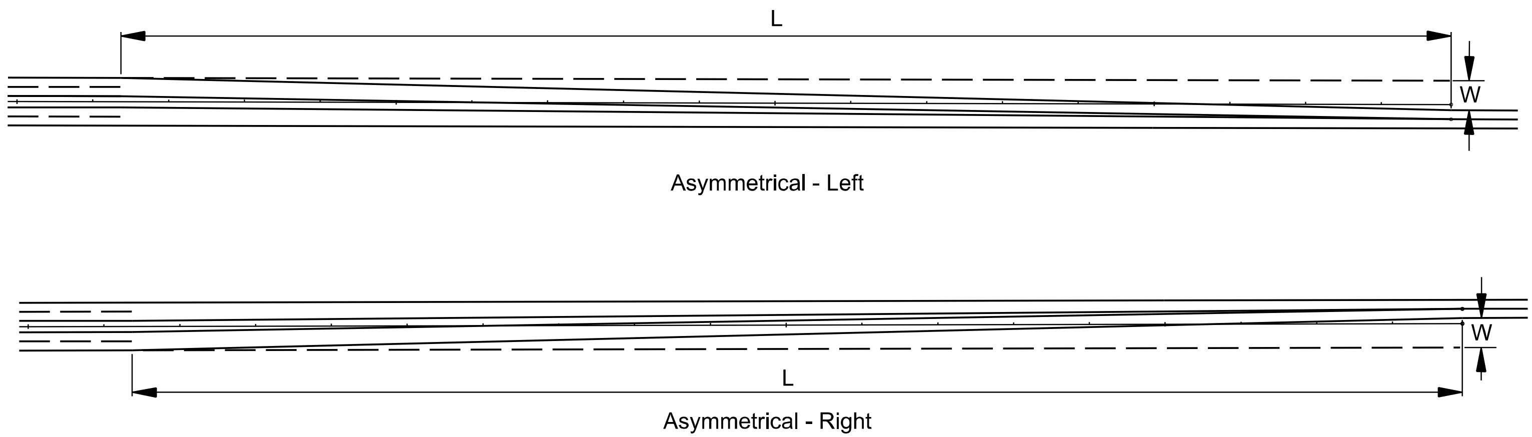
Solution:
$W = 31.5 - 24 = 7.5$ feet
$L = 7.5 \times 40^2/60 = 200$ feet

TWLTL LANE TRANSITION
(Five-Lane TWLT to Four-Lane Section)
Figure 7.4-B



TWLTL LANE TRANSITION
(Five-Lane TWLT to Two-Lane Section)
Figure 7.4-C

TWLTL LANE TRANSITION
(Asymmetrical)
Figure 7.4-D



$L = WS$ ($S > 45$ mph), or $L = WS^2/60$ ($S < 45$ mph)
L = Taper Length, Feet
W = Transition Width, Feet
S = Design Speed, MPH

NOTE: The lane drop taper should be computed based on the equations given. The length for the lane addition taper can be reduced to 50% of the lane drop taper length under constrained conditions.

7.5 BRIDGE AND UNDERPASS CROSS SECTIONS

The roadway cross section should be carried over and under bridges, which often requires special considerations because of the confining nature of bridges and their high unit costs.

7.5.1 Bridges

7.5.1.1 Bridge Roadway Widths

In general, bridge widths should match the approach roadway widths (traveled way plus shoulders). Figure 7.5-A provides guidelines for bridge widths. However, in determining the width for major water crossings, the designer needs to consider the cost of the structure, traffic volumes and potential for future width requirements.

Approach Roadway	Conditions	Bridge Width (Gutter to Gutter)
Urban Streets (Curb and Gutter)	With or without concrete sidewalk	Provide a sidewalk on bridge matching roadway gutter hinge points with bridge gutter hinge points.
Freeways and Arterials	12-foot shoulder (10-foot paved + 2-foot unpaved)	Use 12-foot shoulder hinge point for bridge gutter line.
	10-foot shoulder (paved and unpaved)	Use 10-foot shoulder hinge point for bridge gutter line.
	10-foot shoulder (6-foot paved + 4-foot unpaved)	Use 10-foot shoulder hinge point for bridge gutter line on inside of divided highways.
	10-foot shoulder (4-foot paved + 6-foot unpaved)	
Rural Collectors and Local Roads	6- to 8-foot shoulders (2-foot paved + 4- to 6-foot unpaved) with paved roadway	Use shoulder hinge point for bridge gutter line. Bridge width is equal to width of roadway section (outside shoulder to outside shoulder).
Ramps	In direction of traffic (left) 10-foot shoulder (4-foot paved + 6-foot unpaved)	Use 10-foot shoulder line for bridge gutter line.
	In direction of traffic (right) 10-foot shoulder (6-foot paved + 4-foot unpaved)	
	With curb and gutter	Use the roadway gutter hinge point for bridge gutter line.

GUIDELINES FOR BRIDGE ROADWAY WIDTHS
Figure 7.5-A

7.5.1.2 Vertical Clearance

Establish vertical clearances above all sections of pavement including the shoulder. Section 6.6 and the design tables in Chapters 14 through 17 provide the minimum vertical clearances for new construction and reconstruction projects. For 3R projects, existing vertical clearances may be retained if the structure is not being reconstructed.

7.5.1.3 Highway Grade Separations

Horizontal clearances for highway grade separated structures, where guardrail or barrier protection is not provided, should conform to the clear zone requirements in the AASHTO *Roadside Design Guide*. Clearances may be reduced where protection is provided. These are minimum requirements; the designer should coordinate with the structural designer prior to finalizing the length of the bridge and the horizontal clearances.

7.5.1.4 Highway Overpassing Railroad

The horizontal clearance, measured from the centerline of the track to the face of the adjacent bridge substructure, should be a minimum of 25 feet. The horizontal clearance from the centerline of track to the face of the embankment fill slope, measured to the elevation to the highest rail, should be 20 feet. This 20-foot clearance may be increased at individual structure locations, as required, to provide adequate drainage or to allow adequate room to accommodate other special situations (e.g., future tracks).

When an existing overpass over a railroad is to be widened or rehabilitated, the existing horizontal clearances should be maintained, if less than 25 feet.

7.5.2 Underpasses

The approaching roadway cross section, including any auxiliary lanes, should be carried through the underpass. Desirably, include the clear zone width for each side through the underpass. It is important to consider the potential for further development or traffic increases in the vicinity of the underpass that may significantly increase traffic or pedestrian volumes. Provide sufficient lateral clearance for one additional lane in each direction, if warranted, for future widening.

7.5.3 Traveled Way Width Reductions

When approaching a narrow bridge or underpass, the traveled way width may need to be reduced to allow the roadway to pass over or under a bridge. The Department determines the need for traveled way reductions on a case-by-case basis. Where it is deemed necessary, design the traveled way reduction transitions using the taper rates in Figure 7.5-B.

Design Speed (mph)	**Taper Rate**
30	15:1
35	20:1
40	27:1
45	45:1
50	50:1
55	55:1
60	60:1
65	65:1
70	70:1

Note: *Taper Length (L) = Taper Rate x Offset Distance*

TAPER RATES FOR LANE REDUCTIONS
Figure 7.5-B

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7.6 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. *Guide for the Development of Bicycle Facilities*, AASHTO, 2012.

3. *SCDOT ADA Transition Plan*, SCDOT.

4. *Highway Capacity Manual* 2010, TRB, 2010.

5. *Roadside Design Guide*, AASHTO, 2011.

6. *Low Cost Methods for Improving Traffic Operations on Two-Lane Roads*, Report No. FHWA-IP-87-2.

7. NCHRP Report 375, *Median Intersection Design*, TRB, 1995.

8. NCHRP Report 395, *Capacity and Operational Effects of Midblock Left-Turn Lanes*, TRB, 1997.

9. *Manual on Uniform Traffic Control Devices,* FHWA, ATSSA, AASHTO and ITE, 2009.

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Chapter 8

ROADSIDE SAFETY

SOUTH CAROLINA ROADWAY DESIGN MANUAL

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Chapter 8
ROADSIDE SAFETY

The ideal roadway would be entirely free of any roadside obstructions or other hazardous conditions. This is rarely practical because of natural, economic and environmental factors. The AASHTO *Roadside Design Guide*, *SCDOT Standard Drawings* and SCDOT design memorandum provide the Department's roadside safety criteria.

8.1 CLEAR ZONES

The clear zone widths presented in the AASHTO *Roadside Design Guide* are based on limited empirical data that has been extrapolated to a wide range of conditions. Therefore, the distances imply a degree of accuracy that does not exist. They do, however, provide a good frame of reference for making decisions on providing a safe roadside area. The designer must evaluate each application of the clear zone distance individually, exercising engineering judgment.

When determining clear zone, use the upper range from the chart and include the horizontal curve adjustment factors.

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8.2 REFERENCES

1. *Roadside Design Guide*, AASHTO, 2011.

2. NCHRP 350 *Recommended Procedures for the Safety Performance Evaluation of Highway Features*, Transportation Research Board, 1993.

3. *Manual for Assessing Safety Hardware*, AASHTO, 2016.

4. Memorandum of Agreement for Federal-Aid Preventive Maintenance Projects, May 2015.

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Chapter 9

INTERSECTIONS

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Chapter 9
INTERSECTIONS

This chapter discusses the geometric design of at-grade intersections including intersection alignment and profile, turning radii, turning roadways, turning lanes, channelization, roundabouts, median openings and at-grade railroad crossings. Intersection sight distance is discussed in Chapter 4 “Sight Distance.” The intersection is an important part of the highway system. The operational efficiency, capacity, safety and cost of the system depend largely upon its design, especially in urban areas. The primary objective of intersection design is to reduce potential conflicts between vehicles, bicycles and pedestrians while providing for the convenience, ease and comfort of those traversing the intersection.

9.1 DEFINITIONS

1. Approach. A road providing access from a public way to a highway, street or road to an abutting property.

2. Channelization Islands. Channelization islands control and direct traffic movements and guide the driver into the proper channel. Traffic is generally passing in the same direction on both sides of the island.

3. Channelization. The directing of traffic through an intersection by the use of pavement markings (including striping, raised reflectors, etc.) for divisional or directional islands.

4. Comfort Criteria. Criteria that is based on the comfort effect of change in vertical direction in a sag vertical curve because of the combined gravitational and centrifugal forces.

5. Corner Island. A raised or painted island to channel the right-turn movement.

6. Curb Return. The circular segment of curb at an intersection that connects the tangent portions of the intersecting legs.

7. Design Vehicle. The vehicle used to determine turning radii, off-tracking characteristics, pavement designs, etc., at intersections.

8. Divisional Islands. Divisional islands segregate opposing traffic flows, alert the driver to the cross road ahead and regulate traffic through the intersection.

9. Grade Separation. A crossing of two highways, or a highway and a railroad, at different levels.

10. Interchange. A system of ramps in conjunction with one or more grade separations, providing for the movement of traffic between two or more roadways on different levels.

11. Intersection Sight Distance (ISD). The sight distance required within the corners of intersections to safely allow a variety of vehicular access or crossing maneuvers based on the type of traffic control at the intersection.

12. Intersection. The general area where two or more highways join or cross at grade.

13. Landing Area. The area approaching an intersection for stopping and storage of vehicles.

14. No Control Intersection. An intersection where none of the legs are controlled by a traffic control device.

15. Radius Return. The point along the mainline pavement edge where the curb return of an intersection meets the tangent portion, or a point tangent to a mainline curve.

16. Refuge Island. Corner or divisional islands that function to aid and protect pedestrians and bicyclists who cross a wide roadway.

17. Roundabout. A circular intersection with yield control of all entering traffic, channelized approaches with raised splitter islands, counter-clockwise circulation, and appropriate geometric curvature to encourage a travel speed on the circulating roadway of less than 30 miles per hour.

18. Signalized Intersection. An intersection where all legs are controlled by a traffic signal.

19. Stop-Controlled Intersection. An intersection where one or more legs are controlled by a stop sign.

20. Traffic Circle. A traffic circle is a type of intersection that directs both turning and through traffic onto a one-way circular roadway, usually built for the purposes of traffic calming or aesthetics. Traffic circles are typically not used on the State Highway System.

21. Turn Lane. An auxiliary lane adjoining the through traveled way for speed change, storage and turning.

22. Turning Roadway. A channelized roadway (created by an island) connecting two legs of an at-grade intersection. Interchange ramps are not considered turning roadways.

23. Turning Template. A graphic representation of a design vehicle’s turning path depicting various angles of turns for use in determining acceptable turning radii designs.

24. Yield-Controlled Intersection. An intersection where one or more legs are controlled by a yield sign.

9.2 GENERAL DESIGN CONTROLS

9.2.1 General Design Considerations

In every intersection design, there are many conflicting requirements that must be balanced against each other to produce a safe and efficient design. The basic elements that must be taken into consideration include:

1. Human Factors. These include:

    - driving habits,
    - ability to make decisions,
    - driver expectancy,
    - decision and reaction time,
    - conformance to natural paths of movement,
    - pedestrian use and habits, and
    - bicycle traffic use and habits.

    Intersections should be as simple as practical to minimize the number of possible conflicts and to avoid subsequent confusion and demands on drivers to recognize and rapidly react to complex situations.

2. Traffic Considerations. These include:

    - capacity and actual traffic volumes,
    - AADT and/or DHV,
    - vehicular composition,
    - turning movements,
    - vehicular speeds (design and operating),
    - transit involvement,
    - crash history, and
    - bicycle and pedestrian movements.

    Traffic Engineering will provide the AADT, vehicle composition and DHV, including turning movement volumes. Planning will provide future year projections, including turning movement volumes.

3. Physical Elements. These include:

    - character and use of abutting property,
    - right of way,
    - coordination of vertical profiles of the intersecting roads,
    - coordination of horizontal and vertical alignment for intersections on curves,
    - available sight distance,
    - intersection angle,
    - conflict area,
    - geometric design,
    - channelization,
    - traffic control devices,

- lighting,
- safety features,
- bicycle and pedestrian traffic,
- environmental impact, and
- drainage requirements.

4. Economic Factors. These include:

- cost of improvements,
- effects of controlling access to adjacent property, and
- impact on fuel usage.

5. Consistency. Intersection alignment should be consistent. Avoid sharp curves at the intersection.

6. Functional Intersection Area. An intersection can be defined by both its functional and physical areas. These are illustrated in Figure 9.2-A. The functional area of the intersection extends both upstream and downstream from the physical intersection area and includes any auxiliary lanes and their associated channelization.

   The functional area on the approach to an intersection, shown in Figure 9.2-B, consists of the three basic elements — perception-reaction distance, maneuver distance and queue-storage distance.

7. Physical Intersection Area. The limits or physical area within an intersection is defined by the location of stop bars when present. When stop bars are absent, the limits of or physical area within an intersection is defined by the location points where the corner radii between adjacent roadway approaches tie to the edge of pavement or the edge of travel lane adjacent to the edge of pavement of each roadway approach.

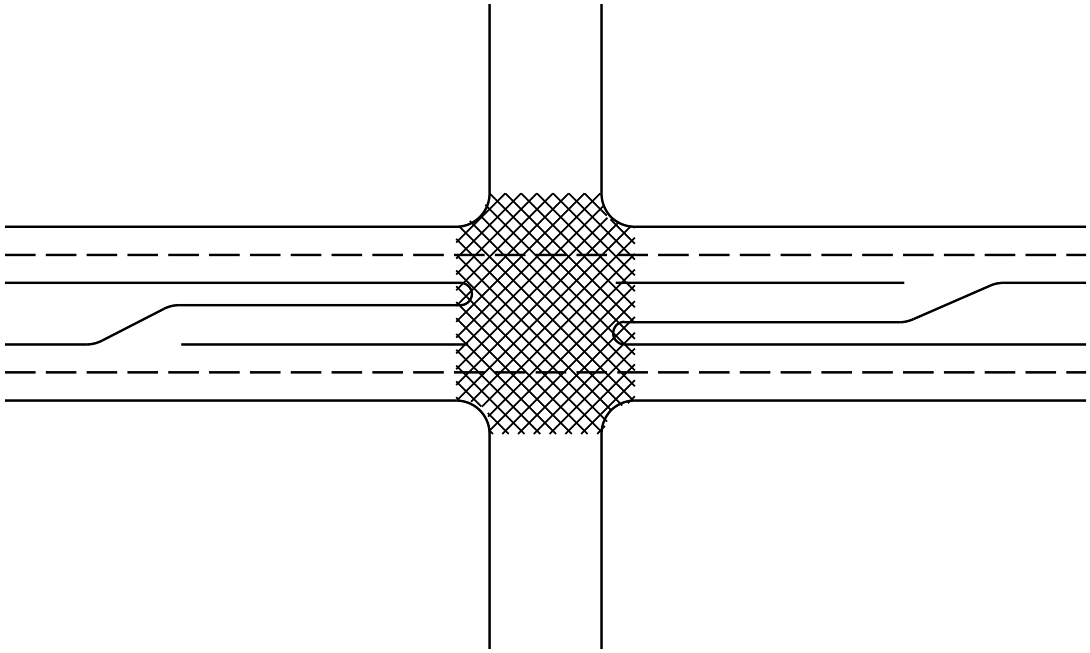
9.2.2 Intersection Types

Intersections are usually a three-leg, four-leg or multi-leg design. Individual intersections may vary in size and shape and may be channelized. The principal design factors that affect the selection of intersection type and its design characteristics are discussed in Section 9.2.1. Selection of the intersection type will be determined on a site-by-site basis.

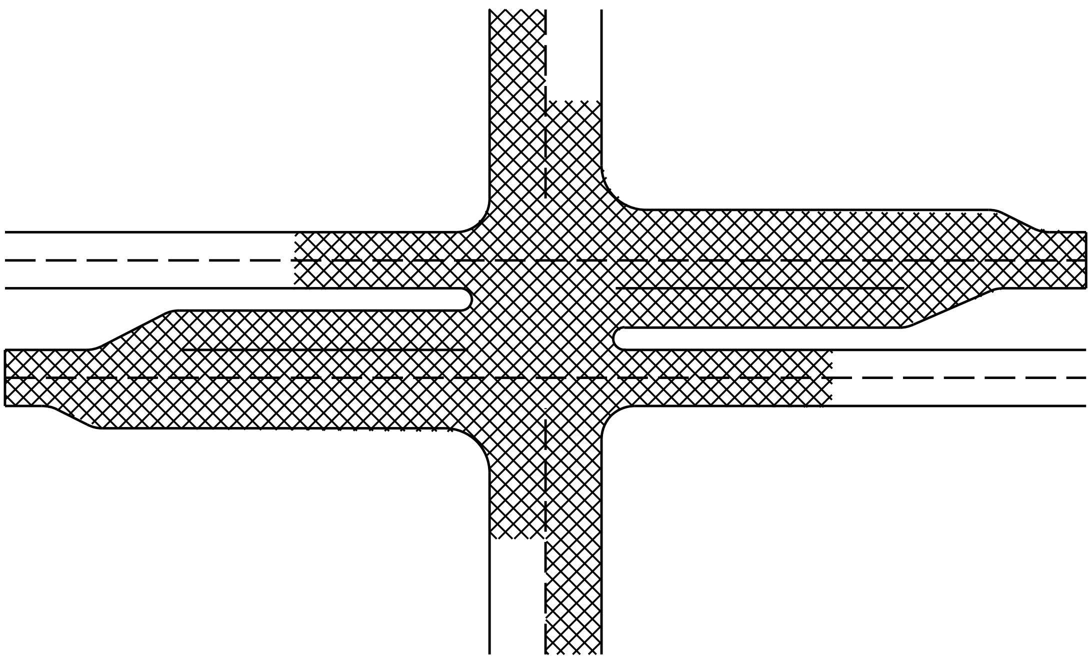
Multi-leg intersections are those with five or more intersection legs. Avoid using multi-leg intersections. Wherever practical, rearrange the legs to remove conflicting movements. This may be accomplished by realigning one or more of the intersecting legs and combining some traffic movements at adjacent subsidiary intersections or providing a roundabout.

A roundabout is a channelized intersection where traffic moves around a center island, counter-clockwise. Section 9.7 provides guidance on the selection and design of roundabouts. The designer should also review NCHRP Report 672, *Roundabouts: An Information Guide*.

Innovative intersections may offer additional benefits as compare to traditional intersections and may be considered on a case-by-case basis. Coordinate these intersection designs with Traffic Engineering.

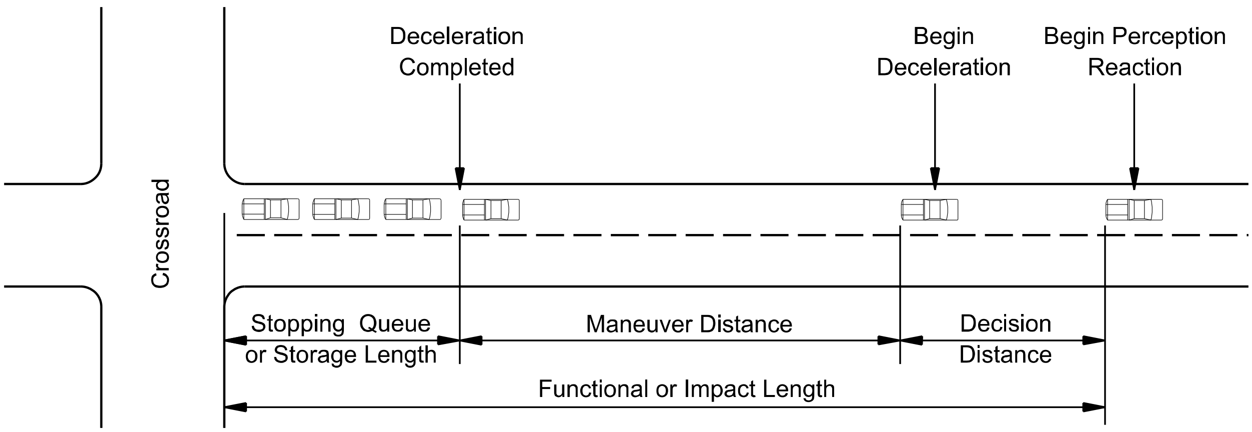


Physical Area



Functional Intersection Area

PHYSICAL AND FUNCTIONAL INTERSECTION AREA
Figure 9.2-A



ELEMENTS OF THE FUNCTIONAL AREA OF AN INTERSECTION
Figure 9.2-B

9.2.3 Intersection Spacing

If practical, avoid short distances between intersections because they tend to impede traffic operations. For example, if two intersections are close together and require signalization, they may need to be considered as one intersection for signalization purposes. To operate safely, each leg of the intersection may require a separate signal phase, thereby greatly reducing the capacity for both intersections. Short spacing between intersections may hinder or even restrict effective left-turn movements. Where practical, realign the roadways to form a single intersection.

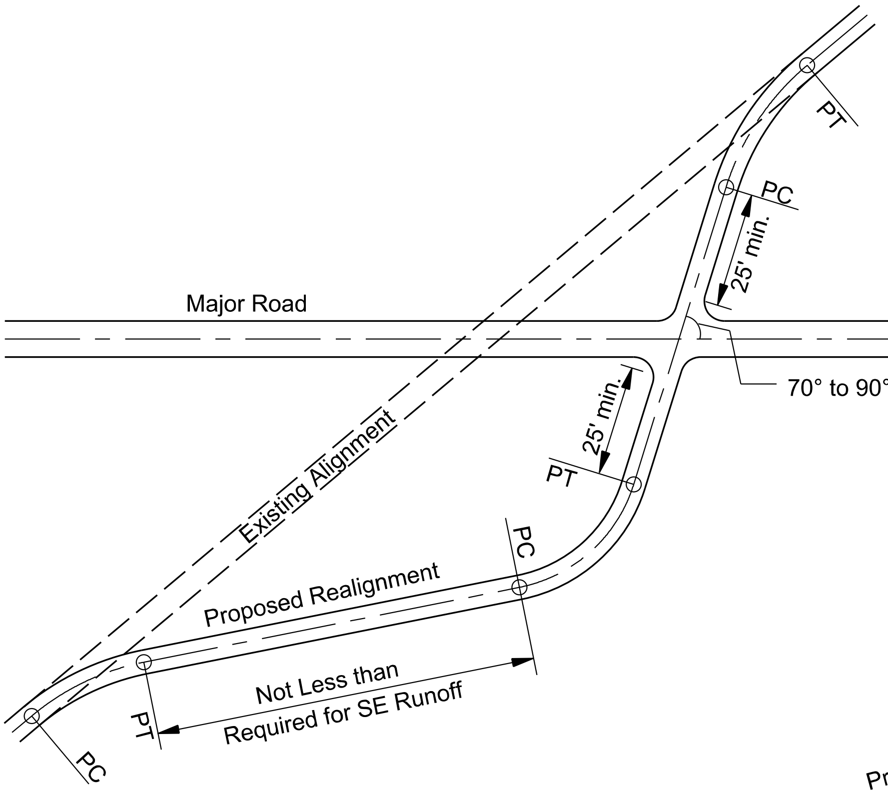
To operate efficiently, urban intersections should be a minimum of 500 feet apart. For rural areas, provide a minimum spacing of ¼ mile and, desirably, ½ mile apart. Generally, treat signalized and unsignalized intersections the same. Because of changing traffic patterns, development and crash concerns, unsignalized intersections may be converted to signalized intersections in the future. Conduct a capacity analysis to evaluate traffic conditions between the intersections.

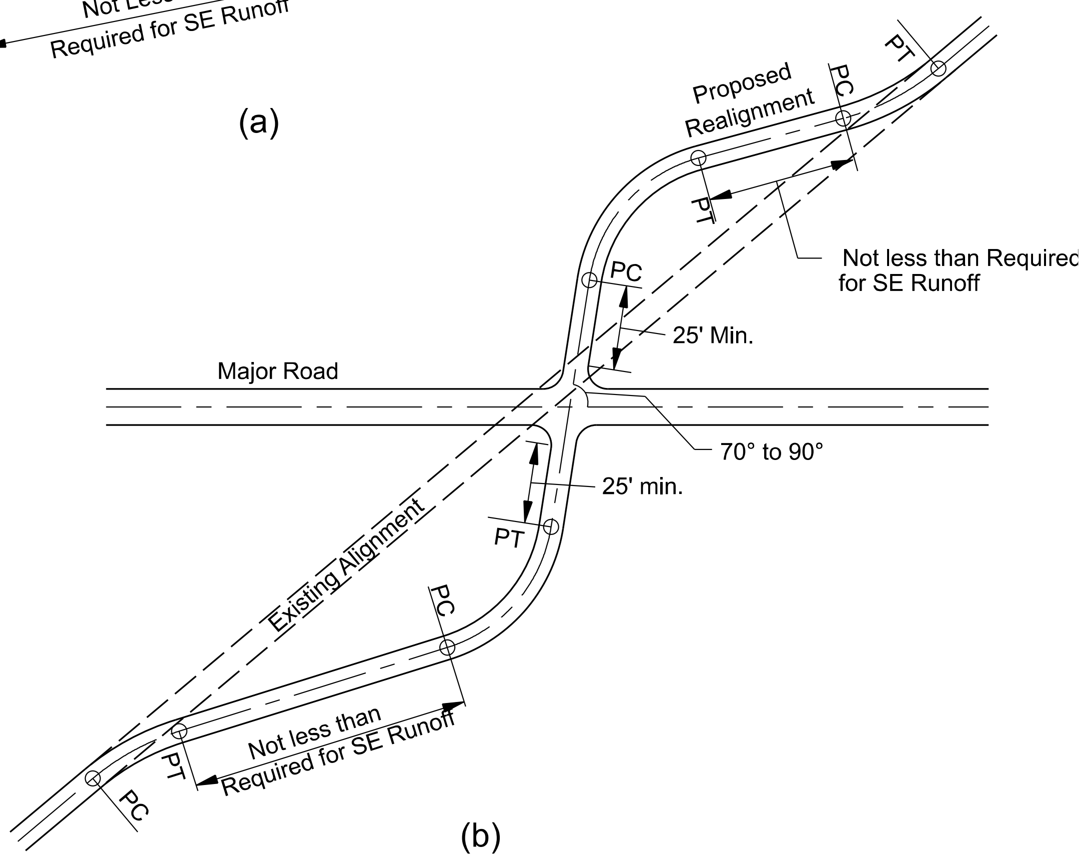
In addition, avoid short gaps between opposing "T" intersections. In general, all new intersections should preferably be at least 500 feet to 700 feet apart. See Section 9.2.6.3 and Figure 9.2-C for additional guidance.

9.2.4 T-Intersections

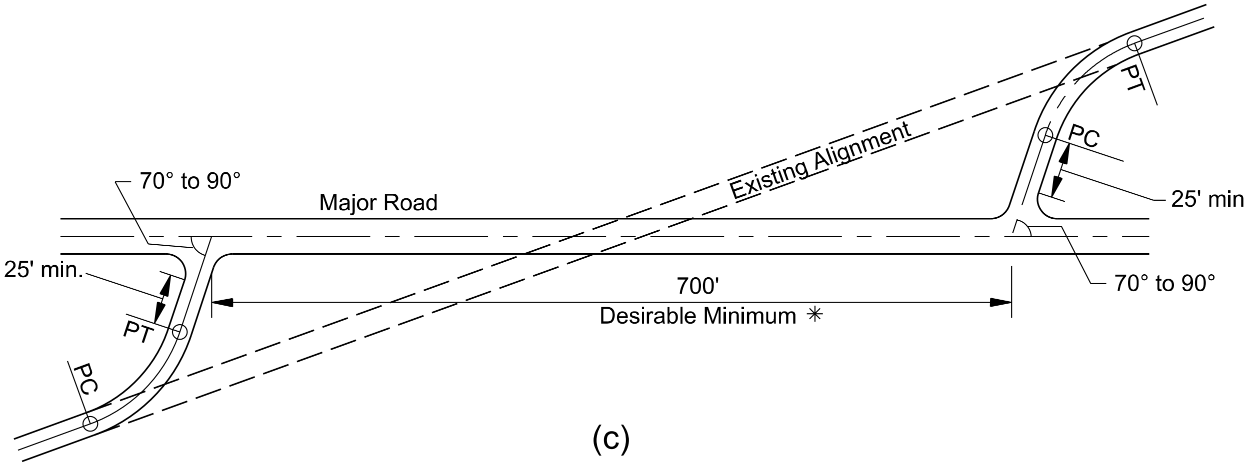
Where a roadway intersects with another road in a T-configuration, consider the following design criteria:

1. Design Speed. Review the approaching roadway's design speed against the stop condition. Typically, a 10 mile per hour difference is preferred between the approaching road design speed and realigned horizontal curve design speed. Greater speed differences may be appropriate depending upon context, impacts and cost associated with the alignment. See Section 9.2.7.3 for guidance on selecting the vertical alignment design speed.





REALIGNMENT OF INTERSECTIONS
Figure 9.2-C



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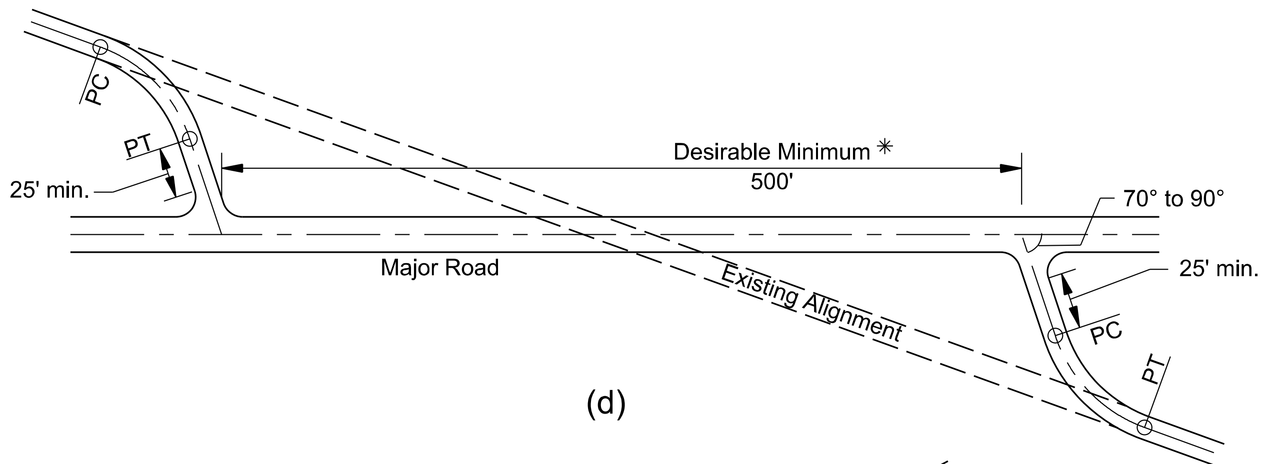

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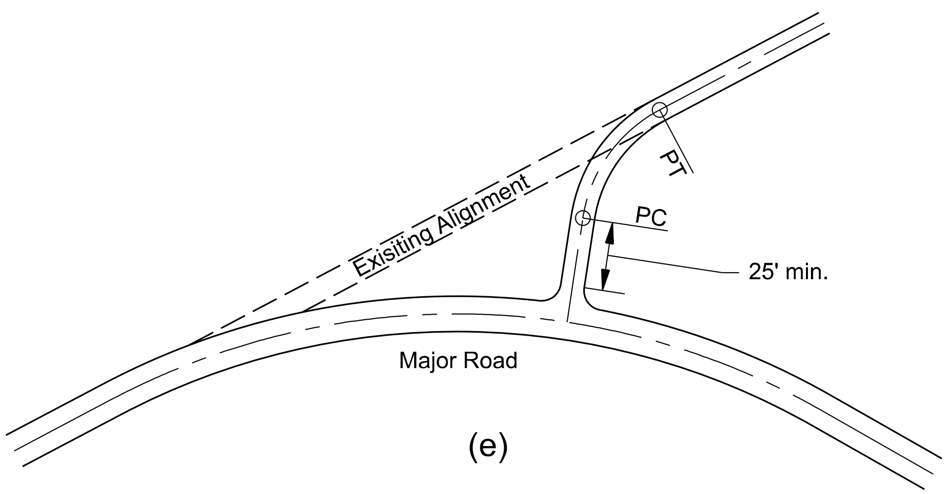


(e)

** Distance between intersections as required by a traffic analysis.*

REALIGNMENT OF INTERSECTIONS
Figure 9.2-C
(Continued)

2. End of Roadway Restriction. If a horizontal curve is too close to the end of the roadway and the recommended superelevation and transition length cannot be obtained, then the horizontal alignment design criteria will not be applicable.

3. No Restrictions. If there is no end of roadway restriction, design the horizontal curve with the proper design speed and transition. Do not mark the plans with "NA."

9.2.5 Design Vehicles

9.2.5.1 Types

The basic design vehicles that may be used for intersection designs in South Carolina include:

- P — Passenger car
- SU-30 — Single-unit truck
- S-BUS-40 — Large school bus
- WB-62 — Tractor/Semitrailer combination
- WB-67 — Tractor/Semitrailer combination
- MH/B — Recreational vehicle, motor home with boat trailer

The AASHTO *A Policy on Geometric Design of Highways and Streets* provides the design details for these vehicles. The turning characteristics of the applicable design vehicle are used to test the adequacy of an existing or proposed design at an intersection. The turning characteristics can be checked with turning templates or with a computer-simulated turning template program.

9.2.5.2 Selection

Figure 9.2-D presents the recommended design vehicles at intersections based on the functional classification of the intersecting highways that the vehicle is turning from and onto. The design vehicles shown in Figure 9.2-D are for new construction and reconstruction projects.

In addition to Figure 9.2-D, use the following guidelines when selecting a design vehicle:

1. Minimum Designs. The SU-30 and/or S-BUS-40 design vehicles are generally the smallest vehicles used in the design of highway intersections. This design reflects that, even in residential areas, garbage trucks, delivery trucks and school buses will be negotiating turns with some frequency. Rural and suburban intersections that serve school bus traffic should, at a minimum, accommodate a turning S-BUS-40 without encroachment outside of the traveled way. Intersections only need to accommodate design vehicles that are expected to use that intersection.

2. Recreational Areas. Recreational areas typically will be designed using the SU-30 design vehicle. This reflects that service vehicles are typically required to maintain the recreational area. Under some circumstances, the motor home with a boat trailer (MH/B) may be the appropriate design vehicle (e.g., campground areas, boat launches).

3. Mixed Use. Some portions of an intersection may be designed with one design vehicle and other portions with another vehicle. For example, it may be desirable to design physical characteristics (e.g., corner islands) for the WB-62 truck, but provide painted channelization for the passenger design vehicle.

For Turn Made		Design Vehicle
From	**Onto**	
Freeway Ramp	Other Facilities	WB-62
Other Facilities	Freeway Ramp	WB-62
Arterial	Arterial Collector Local Local (Residential)	WB-62 WB-62 WB-62 S-BUS-40*
Collector	Arterial Collector Local Local (Residential)	WB-62 WB-62 WB-62 S-BUS-40*
Local	Arterial Collector Local Local (Residential)	WB-62 WB-62 S-BUS-40* S-BUS-40
Local (Residential)	Arterial Collector Local Local (Residential)	S-BUS-40* S-BUS-40* S-BUS-40 S-BUS-40

**With encroachment, a WB-62 vehicle should physically be able to make the turn.*

Note: Use this figure as a guide for new construction and reconstruction projects. Intersections only need to accommodate design vehicles that are expected to use that intersection.

SELECTION OF DESIGN VEHICLE AT INTERSECTIONS
Figure 9.2-D

9.2.6 Intersection Alignment

9.2.6.1 Horizontal Curves

Preferably, an intersection between two roadways should be on tangent sections. Where a minor road intersects a major road on a horizontal curve, the geometric design of the intersection becomes significantly more complicated, particularly for sight distance, turning movements, crossing movements, channelization and superelevation. The following guidelines address horizontal alignment at intersections:

1. Realignment. If relocation of the intersection beyond the horizontal curve is not practical, the designer may be able to realign the minor road to intersect the major road perpendicular to a tangent on the horizontal curve; see Figure 9.2-C(e). Although an

improvement, this arrangement may still result in difficult turning movements due to superelevation on the major road.

2. Superelevation Major Route. If the mainline is on a horizontal curve, minimize the mainline superelevation rate so that slowing or stopped vehicles do not slide across the pavement during wet or icy conditions.

3. Curved Approach. Where a highway or street is on a curved alignment and is approaching a stop condition, the designer needs to give special consideration to the design of the horizontal curvature prior to the intersection.

4. Superelevation Minor Route. If the superelevation development for the selected design speed cannot be obtained due to the alignment, mark the plans "NA" to indicate that the design speed will not be applicable for this situation.

9.2.6.2 Angle of Intersection

The designer should review the crash history prior to designing intersections with skew angles. Highways should intersect at or nearly at right angles. Intersections at acute angles are undesirable because they:

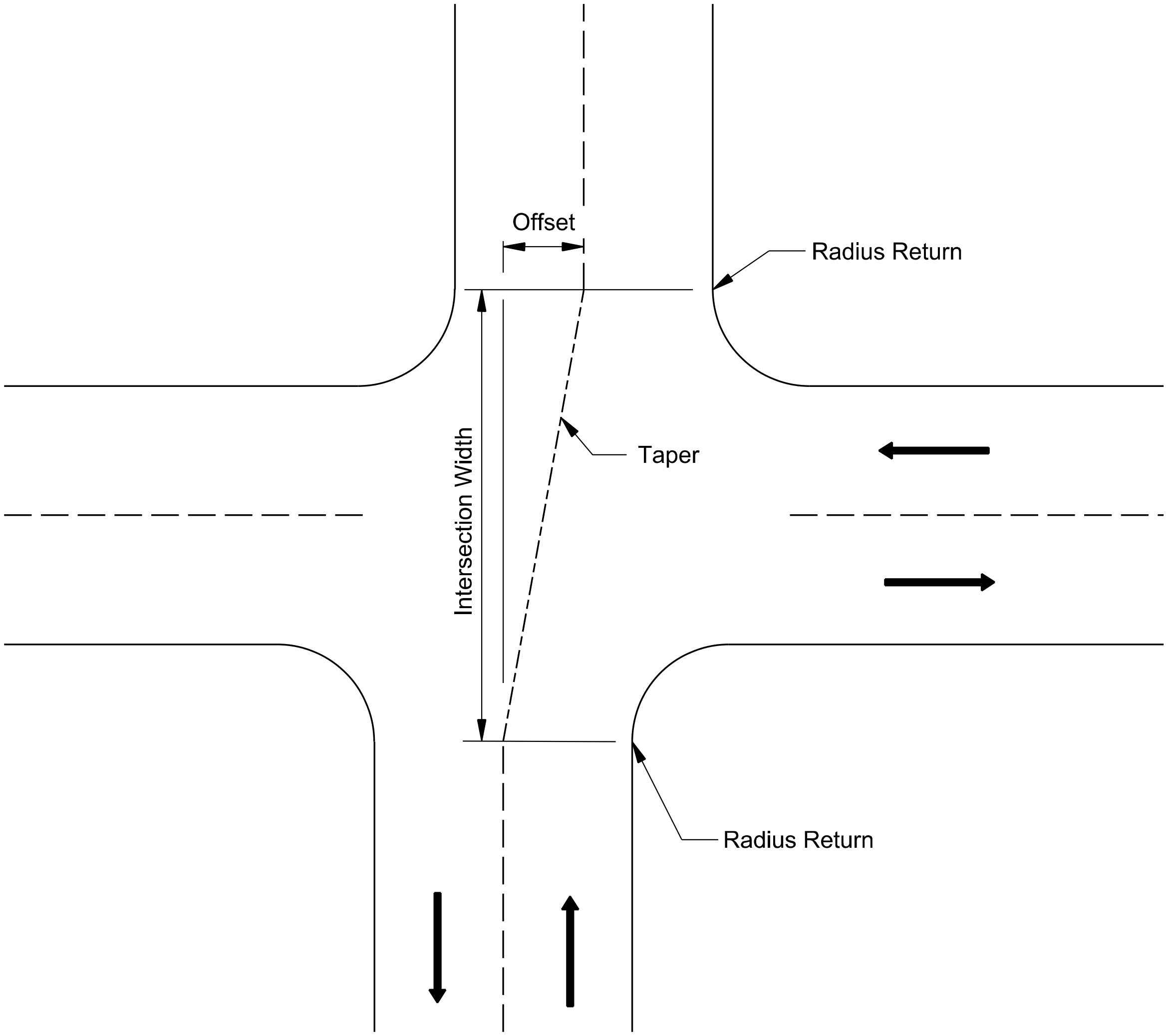
- restrict vehicular turning movements,
- require additional pavement and channelization for large trucks,
- increase exposure time for vehicles and pedestrians crossing the main traffic flow, and
- restrict the crossroad sight distance.

Preferably, the angle of intersection should be perpendicular. Under restricted conditions, where obtaining the right of way to straighten the angle of intersection would be impractical, an intersection angle of 70 degrees may be used. Where turning movements are significantly unbalanced, the intersections may be angled to favor the predominant movement. Intersection angles beyond these ranges may warrant more positive traffic control (e.g., all stop, traffic signals) or geometric improvements (e.g., realignment, greater corner sight distance).

Figure 9.2-C illustrates various angles of intersection and potential improvements that can be made to the alignment. The designer should consider the sight distances at intersection, see Chapter 4 "Sight Distance," and the location of traffic control devices during design to ensure visibility to the traffic control device will meet the *Manual on Uniform Traffic Control Devices* (*MUTCD)* criteria.

9.2.6.3 Offset Intersection Legs

In general, the designer should design four-leg intersections so that opposing approaches line up with each other (i.e., there is no offset between opposing approaches). However, this is not always practical. Figure 9.2-E presents a diagram of an intersection with an offset between opposing approaches. Because of possible conflicts with overlapping turning vehicles, offset intersections should only be allowed to remain on low-volume approaches.



Design Speed (mph)	Allowable Offset (ft)	
	Crossing a 2-Lane Street (1)	Crossing a 5-Lane Street (2)
20(3)	3.7	7.2
25	3.0	5.7
30	2.5	4.8
35	2.1	4.1
40	1.9	3.6
45	1.6	3.2
50	1.5	2.9
55	1.3	2.6
60	1.2	2.4

1. *Assumes a 25-foot corner radius and two 12-foot lanes (i.e., 74 feet).*
2. *Assumes a 40-foot corner radius, four 12-foot lanes and a 15-foot TWLTL (i.e., 143 feet).*
3. *Use the 20-mile per hour design speed for all stopped approaches.*
4. *See discussion in Section 9.2.6.3 for more information.*

PERPENDICULAR OFFSET INTERSECTION LEGS
Figure 9.2-E

The following criteria will apply for offset intersection approaches:

1. Maximum Offset. The maximum offset is determined from the application of a taper equal to V:1 applied to the intersection width, where V is the design speed in miles per hour; see Figure 9.2-E. V is selected as follows:

   - V = 20 miles per hour for stop-controlled approaches.
   - V = the roadway design speed for the free-flowing approaches at a stop-controlled intersection.
   - V = the roadway design speed for the offset approaches at a signalized intersection.

2. Turning Conflicts. Evaluate the entire intersection for conflicts that may result from turning vehicles at an offset intersection. For example, offsets where the jog is to the left may result in significant interference between simultaneous left-turning vehicles.

3. Evaluation Factors. In addition to potential vehicular conflicts, the designer should evaluate the following at existing or proposed offset intersections:

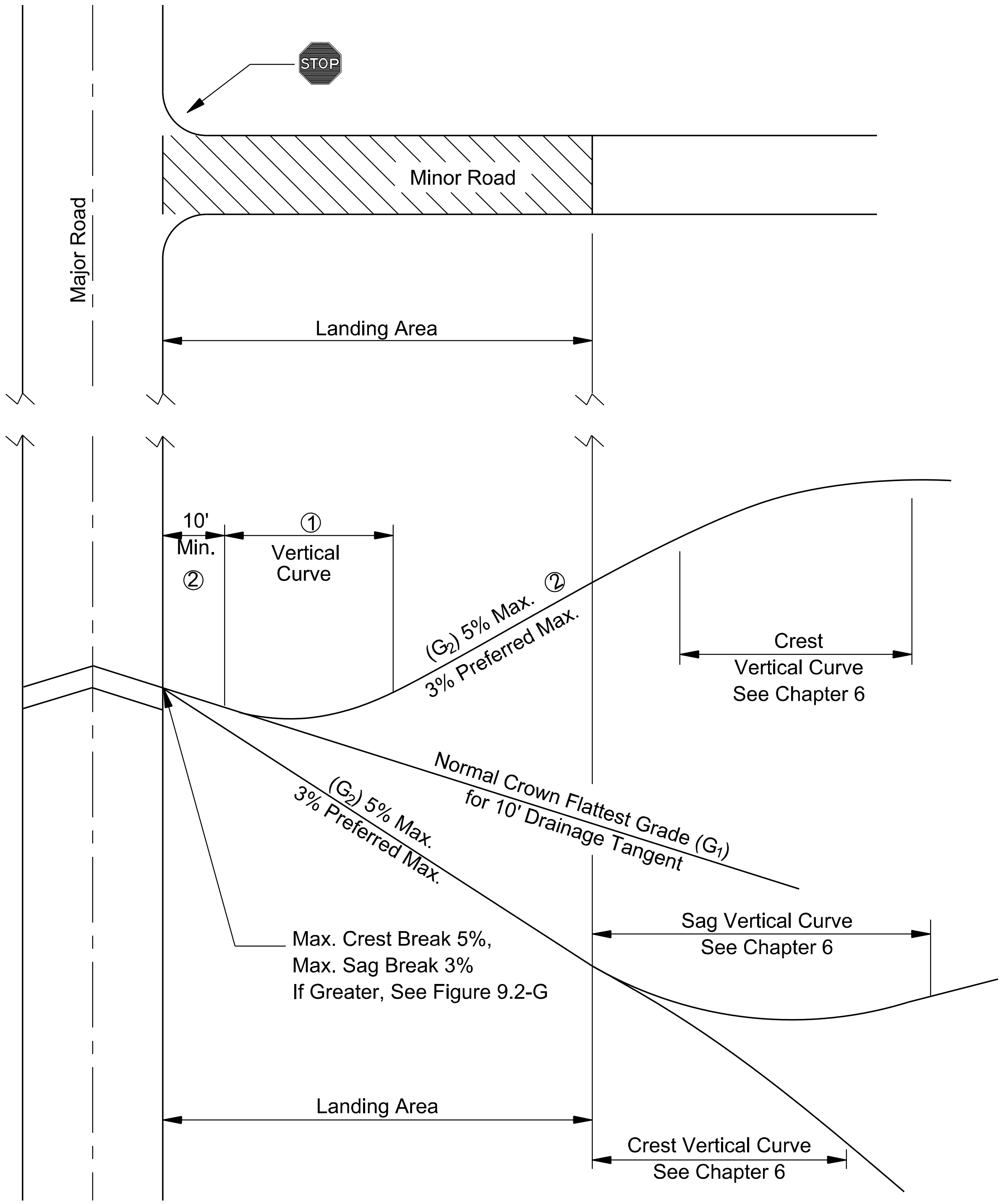
   - through and turning volumes;
   - type of traffic control;
   - impact on all turning maneuvers;
   - intersection geometrics (e.g., sight distance, curb/pavement edge radii); and
   - crash history at existing intersections.

Where existing offset intersections are being considered to remain-in-place, the designer should coordinate the intersection design and traffic control requirements with Traffic Engineering.

9.2.7 Intersection Profiles

9.2.7.1 Gradient

Vertical alignments through intersections should be as flat as practical, but consideration must be given to obtaining positive drainage. Intersection areas or landing areas in the range of 75 feet to 100 feet should be established for minor roads as shown in Figures 9.2-F and 9.2-G. The landing area is the portion of intersecting highways, local roads, and public and private approaches that are used for the storage of stopped vehicles. This landing area should provide for minimum grade changes to provide adequate sight distance and minimize acceleration time for vehicles using the crossroads. Desirably, the landing area will slope away from the intersection on a gradient not to exceed 3 percent, downward or upward. Where the use of grades less than 3 percent may be cost prohibitive, the designer may, with corresponding adjustments to other intersection design elements, use an approach gradient up to 5 percent.



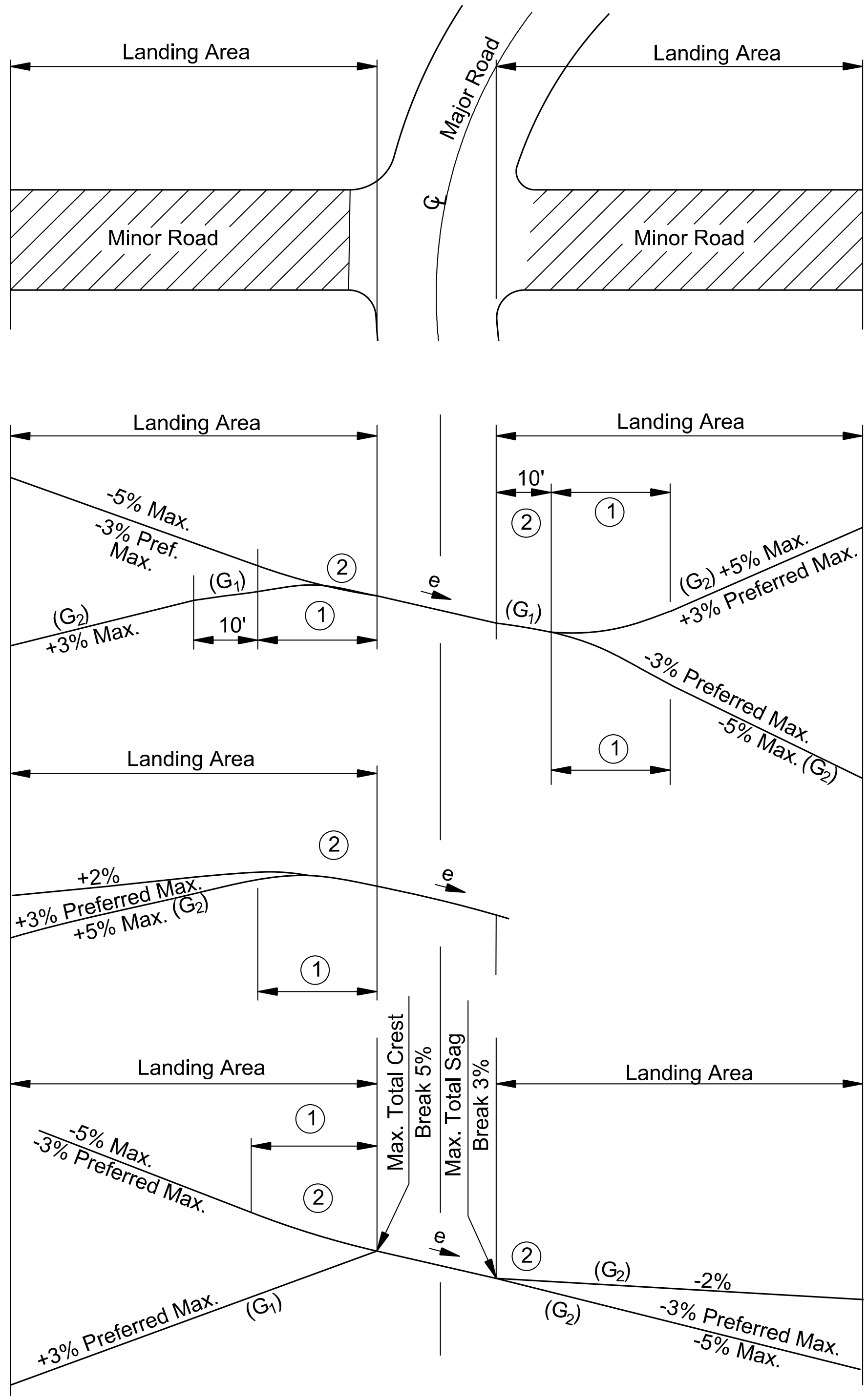
Notes:

① *See Section 9.2.7.3 for the vertical curve options.*

② *If practical, the gradient of the landing area where vehicles may be stored should not exceed 3 percent.*

③ *Actual field conditions will determine the final design.*

VERTICAL PROFILES OF INTERSECTING ROADS
(Tangent Section)
Figure 9.2-F



Notes:

① *See Section 9.2.7.3 for the vertical curve options.*

② *If practical, the gradient of the landing area where vehicles may be stored should not exceed 3 percent.*

③ *Actual field conditions will determine the final design.*

④ *For crest breaks exceeding 5 percent, use a vertical curve with a K-value of 3 or greater. For sag breaks exceeding 3 percent, round the break using a K-value of 5 or greater.*

VERTICAL PROFILE OF INTERSECTING ROADS
(Superelevated Section)
Figure 9.2-G

9.2.7.2 Cross Slope Transitions

One or both of the roadways approaching the intersection may need to be transitioned (or warped) to match or coordinate the cross slope and grade at the intersection. The designer should consider the following:

1. Stop Controlled. When the crossroad is stop controlled, maintain the profile and cross section of the major road through the intersection and transition the cross slopes of the stop-controlled roadway to match the major road cross slope and profile.

2. Signalized Intersection. At signalized intersections or potential signalized intersections, transition the cross slope of the crossroad to meet the profile and cross slope of the major road. If both intersecting roads have approximately equal importance, the designer may want to consider transitioning both roadways to form a plane section through the intersection. Where compromises are necessary between the two major roadways, provide the smoother riding characteristics to the roadway with the higher travel speeds.

3. Transition Distance. In rural areas, transitioning from the normal crown to a warped section should be accomplished in a distance of 50 feet. The 50-foot transition length is also desirable for urban areas but, at a minimum, the transition may be accomplished within the curb return.

9.2.7.3 Vertical Profile

Where the profile of the crossroad is adjusted to meet the major road, this will result in angular breaks for traffic on the crossroad if no vertical curve is inserted. The designer should review the following options for removing the angular breaks; see Figures 9.2-F and 9.2-G:

1. Vertical Curves (SSD). Desirably, use vertical curves through an intersection that meet the criteria for stopping sight distance as described in Chapter 6 “Vertical Alignment.” For stop-controlled legs, design the approach landing vertical curve with a 15 to 30 mile-per-hour design speed. At free-flowing legs and at all legs of a signalized or proposed future signalized intersection, use the design speed of the roadway to design the vertical curve. The grades of the tangents for the vertical curve are the grade of the landing area (i.e., the cross slope of the major roadway) (G_1) and the profile grade of the crossroad (G_2); see Figures 9.2-F and 9.2-G. Locate the Point of Vertical Tangency (PVT) a minimum of 10 feet from the major road traveled way. See Section 6.5.

2. Sag Vertical Curves (Comfort). If lighting is provided at sag vertical curves, a design to the driver comfort criteria may be adequate; see Section 6.5.2.

9.2.7.4 Drainage

Evaluate the profile and transitions at all intersections for impacts on drainage. This is especially important for channelized intersections on curves and grades. This will require the designer to check superelevation transition lengths to ensure that flat sections are minimized. Low points on approach roadway profiles should be beyond a raised corner island to prevent water from being trapped and causing ponding.

9.2.8 Traffic Analysis

A traffic analysis should be conducted before performing the detailed design of any intersection. This analysis will influence several geometric design features including the number of approach lanes, turning lanes, lane widths, channelization and number of departure lanes. The designer should select a level of service (LOS) and future design year, typically 20 years for new construction and major reconstruction projects. If the intersection is within the limits of a future project, the design year for the intersection should be the same as that for the project.

LOS recommendations are provided in the geometric design tables in Chapters 14 through 17. Once the LOS and design traffic volumes are determined, the designer should use the *Highway Capacity Manual* for the detailed capacity analysis. On request, the traffic designer may perform this analysis.

The traffic designer will provide a crash history analysis. Review the analysis to determine if any mitigation techniques should be employed.

9.2.9 Signalized Intersections/Traffic Control

Traffic control signs and signals are devices that control the movement of vehicles and pedestrians at intersections by assigning the right of way to various movements. Traffic signal control provides a key element in controlling the flow of traffic and movement of pedestrians on many urban streets and some rural intersections. For this reason, the proposed signal operation for each intersection of a planned highway or street improvement should be integrated into the design process in order to achieve optimum operational efficiency. The designer should give careful consideration during the plan development phase of a proposed highway improvement to:

- intersection and access locations;
- horizontal and vertical alignment with respect to signal visibility;
- pedestrian requirements; and
- signal operations, including signal phasing and signal coordination (within signal system).

When stop-controlled intersections reach a certain stage of congestion or have severe operational and/or safety problems, it usually becomes necessary to provide traffic signals. The selection and use of this traffic control device should be preceded by a thorough engineering study of roadway and traffic conditions. Traffic signal warrants are discussed in the *MUTCD*.

Lane arrangement is a key factor to the successful operation of signalized intersections. In general, traffic signals are not installed without left-turn lanes on all approaches. The need for providing separate left- or right-turn lanes, to improve signal efficiency, should be evaluated concurrently with the potential for obtaining the required right of way for these lanes, particularly in built-up urban areas.

The *MUTCD* and the *Highway Capacity Manual* provide detailed guidance regarding traffic control signals. There are also a number of computerized programs that address traffic signalization.

Traffic Engineering and the District Traffic Engineers provide the Department's primary expertise regarding the need, installation and maintenance of traffic control signals and traffic control systems. For guidance on traffic signal designs, see the SCDOT *Signal Design Guidelines* or contact Traffic Engineering.

9.2.10 Intersection Sight Distance

In general, intersection sight distance (ISD) refers to the corner sight distance available in intersection quadrants that allows a driver approaching an intersection to observe the actions of vehicles on the crossing leg(s). ISD evaluations involve establishing the needed sight triangle in each quadrant by determining the legs of the triangle on the two intersecting roadways. The necessary clear sight triangle is based on the type of traffic control at the intersection and on the design speeds of the two roadways. For guidance on ISD, see Section 4.4.

The designer should evaluate the effect the intersection profile and alignment will have on ISD. Landings with steep upgrades may place the driver's eye below or in line with roadway appurtenances (e.g., guardrail, signs, traffic control box, signal poles). Also, large skewed intersections will require drivers to look back over their shoulder. The ISD triangular area does not include additional right of way that may be needed for placement of traffic control devices.

9.3 TURNING RADII

9.3.1 Types of Corner Radii

At intersections, the designer must determine how best to accommodate right-turning vehicles. A design must be selected for the edge of pavement or curb lines, which may be one of the following types:

- simple radius,
- simple radius with entering and/or exiting taper(s), or
- compound curve (2 or 3 centered).

Each basic design type has its advantages and disadvantages. The simple radius is the easiest to design and construct and, therefore, it is the most common. However, the designer should also consider the benefits of a simple radius with tapers or compound radii. Their advantages as compared to simple radius designs include:

1. To accommodate a specific vehicle with no encroachment, a simple radius requires greater intersection pavement area than compound curves or a radius with tapers. For large vehicles, a simple radius is often an unreasonable design, unless a channelized island is used.

2. A simple radius results in greater distances for pedestrians to cross than compound curves or a radius with tapers.

3. For angles of turn greater than 90 degrees, a radius with tapers or compound curves are better designs than a simple radius, primarily because less pavement area is required.

9.3.2 Right-Turn Designs

9.3.2.1 Design Considerations

Figure 9.3-A provides the advantages and disadvantages for typical right-turn designs. The following presents several basic parameters the designer should evaluate in determining the proper pavement edge/curb line for turns at intersections:

1. Design Vehicle. In general, select the design vehicle based on the largest vehicle that will use the intersection with some frequency. Section 9.2.5 lists the various design vehicles used by the Department. Figure 9.2-D presents the suggested design vehicle based on the functional classification of the intersecting highways that the vehicle is turning from and onto.

2. Inside Clearance. The selected design vehicle will make the right turn while maintaining a minimum 2-foot clearance from the traveled way, curb line or median.

3. Encroachment. Desirably, do not allow encroachments by the design vehicle into opposing or adjacent lanes while making the right turn. However, this is not always practical and/or cost effective. The designer must evaluate possible encroachments against the construction and right of way impacts. If these impacts are significant and if

Right-turn lane with a lane line pavement marking	Advantages: • Allows right-turn-on-red (unless prohibited), reducing right-turn queues. • Removes turning vehicles from through-vehicle lane for improved intersection operations. • Lower turning speeds provide a safer pedestrian environment. Disadvantages: • All vehicles must stop on red, potentially increasing the right-turn queue. • The absence of an island eliminates its use for: o placement of traffic control devices, and o a pedestrian refuge.
Shared lane with island (Also called slip ramp or free-flow right turn)	Advantages: • Provision of islands permits its use for placement of traffic control devices or as a pedestrian refuge. • Removes turning vehicles from head of queue. Disadvantages: • May encourage higher motorist speeds, which may present a hazard to pedestrians. • If signal support is located on island, pedestrians will need to cross uncontrolled lane to reach pedestrian push button. • The through movement queue may obstruct the throat of the right-turn lane, reducing capacity of the intersection. • Driver attention is split between looking back to merging traffic and looking forward to pedestrian crossing points that may be present in front of the vehicle.
Right-turn lane with island (Also called channelized right-turn lane or free-flow right turn)	Advantages: • Provides relatively free movement for vehicles after yielding to pedestrians and opposing traffic, thus reducing right-turn queues, lowering emissions and increasing capacity. • Provision of islands permits its use for placement of traffic control devices or as a pedestrian refuge. • Removes turning vehicles from the through vehicle lane for improved intersection operations. Disadvantages: • Same as shared lane with island.
Right-turn lane with island and dedicated downstream lane	Advantages: • Provides relatively free movement for vehicles after yielding to pedestrians, thus reducing right-turn queues, lowering emissions and increasing capacity. • Provision of islands permits its use for placement of traffic control devices or as a pedestrian refuge. • Eliminates need to look for merging vehicles (attention may be focused ahead of vehicle because driver is entering dedicated lane). Disadvantages: • Same as shared lane with island. • Vehicles are observed to frequently stop prior to entering the cross street even with an available dedicated lane, because drivers do not know they have a dedicated lane or its length. • Dedicated downstream lane must be sufficient length for vehicles to merge. • Access needs to be managed along dedicated downstream lane to ensure proper operation.

RIGHT-TURN DESIGNS
Figure 9.3-A

through and/or turning volumes are relatively low, the designer may consider accepting some encroachment of the design vehicles into opposing lanes and, in particular, onto local streets. If there are two or more lanes of traffic in the same direction on the road onto which the turn is made, the selected design vehicle can occupy both travel lanes.

4. Parking Lanes in Urban Areas. At many urban intersections, parking lanes will be available on one or both approach legs. This additional width will greatly ease the turning problems for large vehicles at intersections with small curb radii. The designer should coordinate the location of parking to ensure a sufficient distance is available from the intersection to allow the design vehicle to make the right or left turn. The designer should use the applicable turning template to determine this distance.

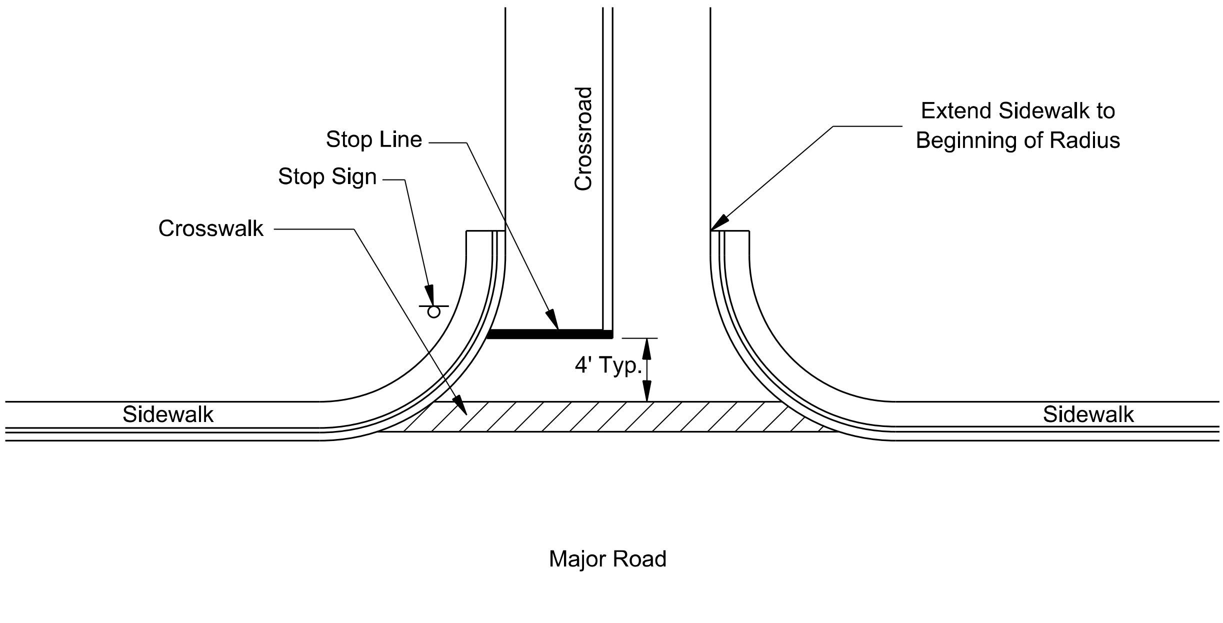
5. Pedestrians. The greater the turning radius, the farther pedestrians must walk across the roadway. This is especially important to slower moving pedestrians. Larger radii also make it more difficult for drivers to see pedestrians. Therefore, the designer should consider pedestrian usage when determining the edge of pavement or curb line design. At intersections, it is important to extend the sidewalk to the beginning of the corner radius on the cross roadway to encourage safe crossings by pedestrians. This extension is shown in Figure 9.3-B. Note that the extension of the sidewalk in Illustration (b) in Figure 9.3-B is extended beyond the radius to match the opposite corner. Discuss sidewalk termini with the traffic designer during the Design Field Review.

6. Radii Design. Once the designer has determined the basic turning parameters (e.g., design vehicle, encroachment, inside clearance), it is necessary to select a type of turning design for the curb return or pavement edge that will meet these criteria and will fit the intersection constraints. Section 9.3.1 discusses the various radii designs used by SCDOT. The radii selection will be determined on a case-by-case basis.

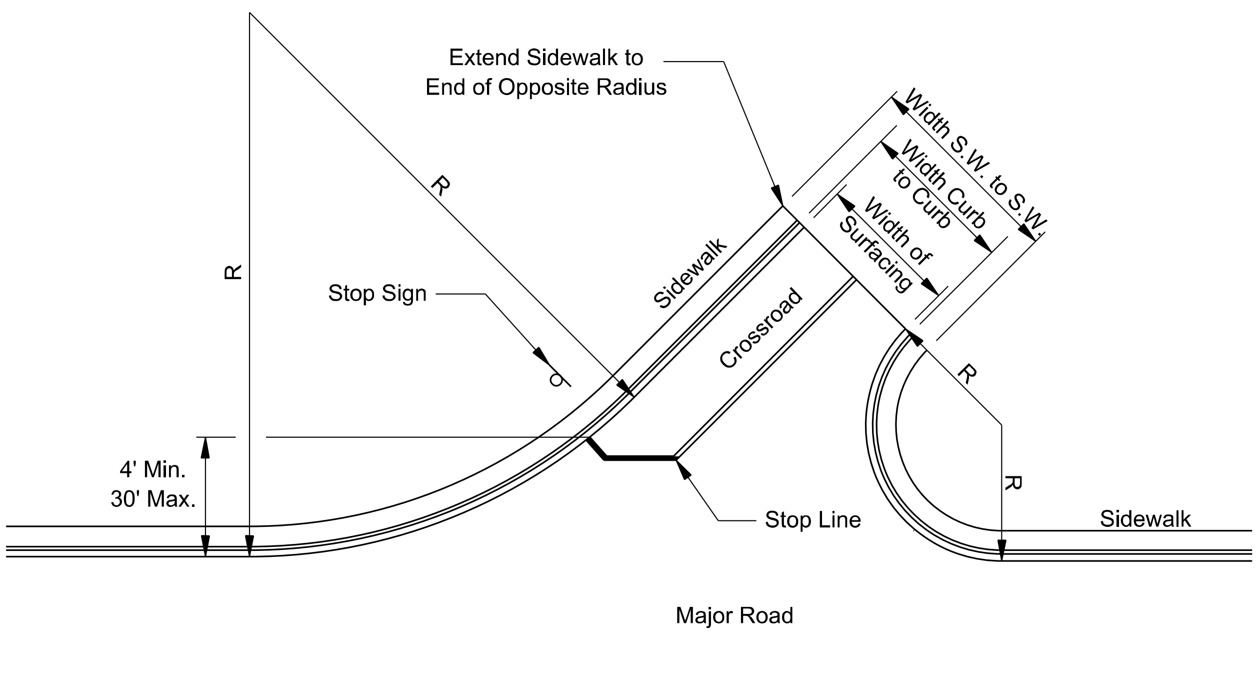
7. Stop Sign and Stop Line Location. When determining the corner radii design, the designer should also consider how the intersection design will affect the placement of the STOP sign and stop line. In placing a STOP sign, visibility of the sign by the approaching vehicles is the primary concern. The stop line indicates the placement of stopped vehicles on the intersection approach, which impacts sight distance and sight lines. In general, traffic safety and operations are improved the closer the stop sign is to the approaching street alignment and the closer the stop line is to the intersecting roadway while still providing for the design vehicle turns. The MUTCD provides guidance and standards on STOP sign and stop line use and placement. See also the *SCDOT Standard Drawings*.

8. Turning Template. The designer should check the design with the applicable turning template or with a computer-simulated turning template program.

Figure 9.3-C illustrates the factors the designer should evaluate in determining the proper design for right turns at intersections. For information on dual-turns lanes, see Section 9.5.4.

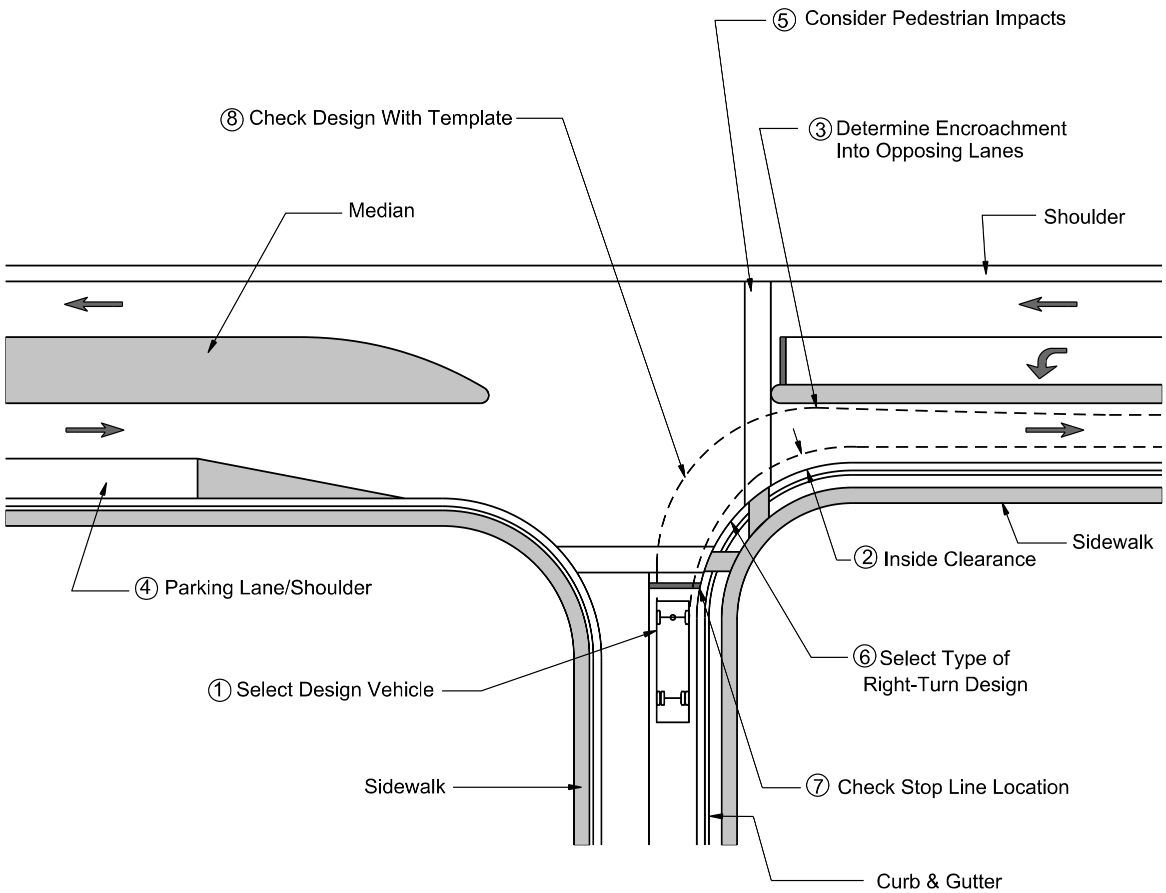


(a) Perpendicular Intersection



(b) Angled Intersection

SIDEWALKS AT INTERSECTIONS
Figure 9.3-B



Note: Numbers apply to the items listed in Section 9.3.2.1.

TURNING RADII DESIGN SUMMARY
Figure 9.3-C

9.3.2.2 Corner Radii Selection Guidelines

In summary, design the corner radii considering the design vehicle, right of way, angle of intersection, number of pedestrians, width and number of lanes on the intersecting streets, stop line location and turning speeds. The following are additional guidelines to consider:

1. Urban Intersections. Provide simple curve corner radii of 40 feet or more at major cross streets so trucks may turn with minimal encroachment. Use turning templates to verify the adequacy of the corner radii.

2. Trucks. Where large trucks and buses are frequent turning vehicles, provide radii based on turning templates. These radii should preferably be two- or three-centered compound curves or simple curves with tapers. Typical radii and tapers are provided in the AASHTO *A Policy on the Geometric Design of Highways and Streets*.

3. Pedestrians. Coordinate radii designs with the crosswalk alignments or provide special designs for pedestrians (e.g., refuge islands, curb ramps).

9.3.3 Left-Turn Designs

Simple curves are typically used for left-turn designs. Occasionally, a two-centered curve is desirable to accommodate the off-tracking of large vehicles provided that the second curve has a larger radius.

The design values for left-turn control radii are a function of the design vehicle (off-tracking), angle of intersection, number of lanes and median width. For roadways intersecting at approximately 90 degrees, radii of 40 to 75 feet will typically satisfy all controlling factors. The criteria for clearance offsets, encroachment, etc., listed in Section 9.3.2 are also applicable to left-turn designs.

Use turning templates to ensure there is no overlap between turning vehicles. Provide 10 feet of offset between opposing left turning wheel paths. If the intersection cannot be modified to avoid overlap, operational considerations can be used to mitigate the overlap.

For information on dual left turns, see Section 9.5.4.

9.4 TURNING ROADWAYS WITH CORNER ISLAND

Where the inner edges of pavements for right turns at intersections are designed to accommodate tractor/semi-trailer combinations or where the desired design permits passenger vehicles to turn at speeds of 10 miles per hour or greater, the pavement area at the corner of the intersection may become excessively large for proper control of traffic. To avoid this, a corner triangular island is used and the connecting roadway between the two intersection legs is defined as a turning roadway. The ramp proper at interchanges is not considered a turning roadway.

9.4.1 Types of Turning Roadways

There are two types of turning roadways:

1. Turning Roadways with Corner Islands. These turning roads are commonly used at intersections where the turning radii design is excessively large such that the pavement area no longer provides the proper traffic control. A painted or raised island is provided to guide the motorist through the turning movement.

2. Free-Flow Turning Roadways. Free-flow turning roadways allow the motorist to move from one roadway to another at moderate to high speeds. Consequently, they often require superelevation, acceleration/deceleration lanes, etc. Because of the large area required for these types of roadways, the Department rarely uses these types of turning roadways. The designer is referred to the AASHTO *Policy on Geometric Design of Highways and Streets* for the necessary design details for free-flow turning roadways.

9.4.2 Guidelines

The need for a turning roadway will be determined on a case-by-case basis. The designer should consider the following guidelines in determining the need for a turning roadway:

1. Trucks. A turning roadway is usually required when the selected design vehicle is a tractor/semitrailer combination.

2. Island Type and Size. The recommended minimum size is 50 square feet (urban) and 100 square feet (rural). Desirably, all triangular islands will be at least 100 square feet, if practical. Islands used for pedestrian refuge should be at least 150 square feet to allow for the construction of curb ramps or channels for the disabled.

3. Level of Service. A turning roadway can often improve the level of service (LOS) for that movement through the intersection. At signalized intersections, a turning roadway may significantly improve the operation of the intersection by removing the right-turning vehicles from the signal timing. Level-of-service criteria are provided in the geometric design tables in Chapters 14 through 16.

4. Crashes. Consider using a turning roadway with a right-turn lane if there are significant numbers of rear-end type crashes at an intersection. Turning roadways with larger radii, in conjunction with a right-turn lane, will allow vehicles to make the turning movements at higher speeds and, consequently, should reduce these types of crashes. Additionally,

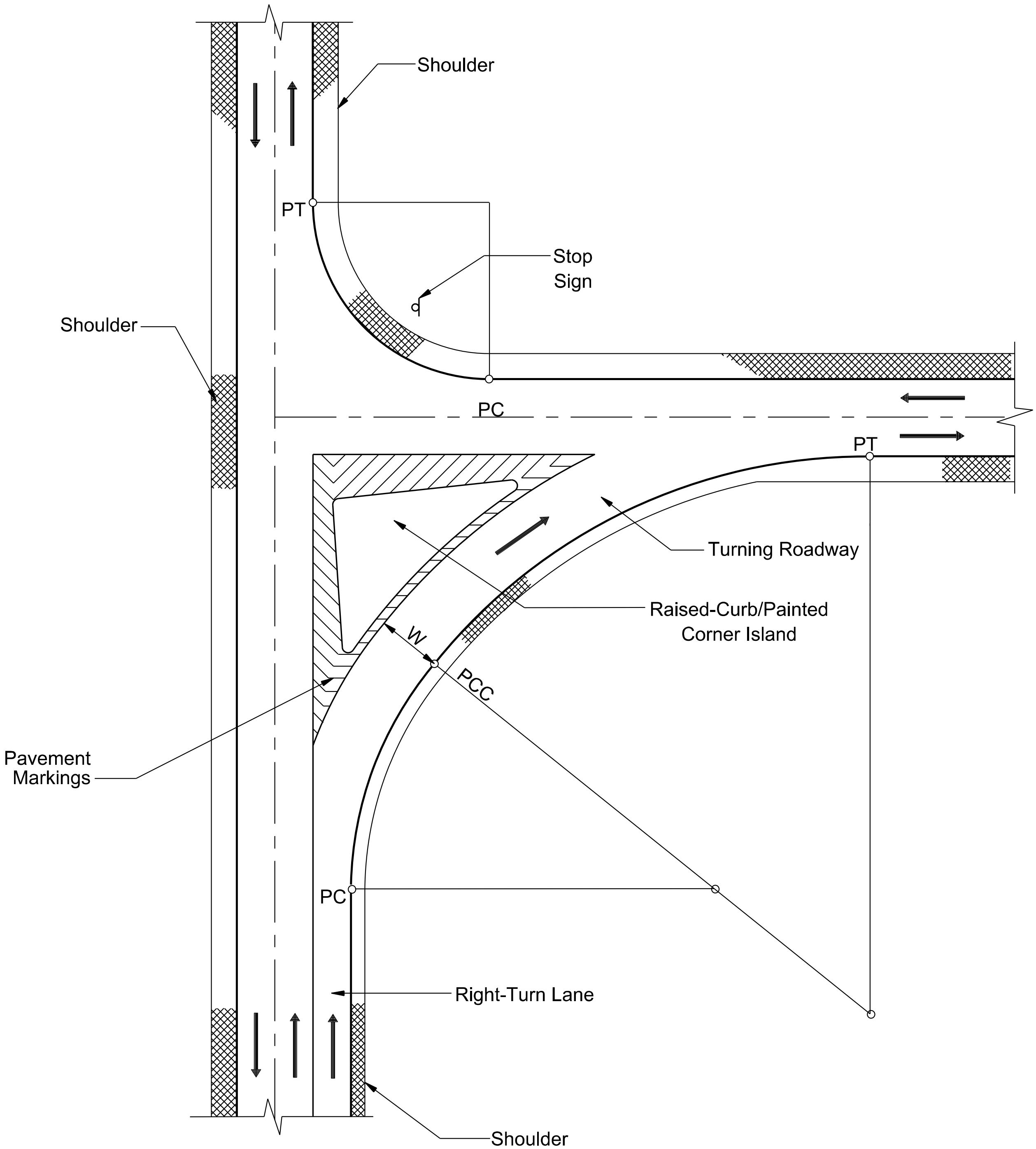
acceleration and/or deceleration lanes should be considered in conjunction with the turning roadway.

9.4.3 Design Criteria

Figure 9.4-A illustrates a typical design for a turning roadway. Section 9.4.4 provides the criteria for turning roadway widths. Where free-flow turning roadways are provided, see the AASHTO *Policy on the Geometric Design of Highways and Streets* for design details on design speed, superelevation, horizontal alignment, acceleration/deceleration lanes, etc.

9.4.4 Width

Turning roadway widths are dependent upon the turning radii and design vehicle selected. Figure 9.2-D provides the criteria for selection of the appropriate design vehicle. Figure 9.4-B presents the turning roadway pavement widths for various design vehicles based on one-lane, one-way operation with no provision for passing a stalled vehicle. This design is generally appropriate for most at-grade intersections. The pavement widths in Figure 9.4-B provide an extra 6 feet of clearance beyond the design vehicle's swept path. This additional width provides extra room for maneuverability, driver variances and the occasional larger vehicle.



Notes:

1. *W = Width of turning roadway; see Figure 9.4-B.*
2. *See Figure 9.6-B for details on the corner island designs.*

TYPICAL TURNING ROADWAY LAYOUT
Figure 9.4-A

Radius on Inner Edge of Pavement R (ft)	Case 1, One-Lane, One-Way Operation, No Provision for Passing a Stalled Vehicle (ft)			
	P	S-BUS-40	WB-62	MH/B
50	13	18	44	21
75	13	17	30	19
100	13	16	25	17
150	12	15	22	16
200	12	15	20	16
300	12	15	18	15
400	12	15	17	15
500	12	14	17	15
Tangent	12	14	15	14

Notes:

1. *If vertical curb is used on one side, then add 1 foot for the curb offset to the table value.*
2. *If vertical curb is used on both sides, then add 2 feet (1 foot on each side) for the curb offset to the table value.*
3. *Only use the turning roadway widths in this figure as a guide, and check the final design with the applicable turning template or computer-simulated turning template program.*

PAVEMENT WIDTHS FOR TURNING ROADWAYS WITH CORNER ISLANDS
Figure 9.4-B

9.5 AUXILIARY TURN LANES

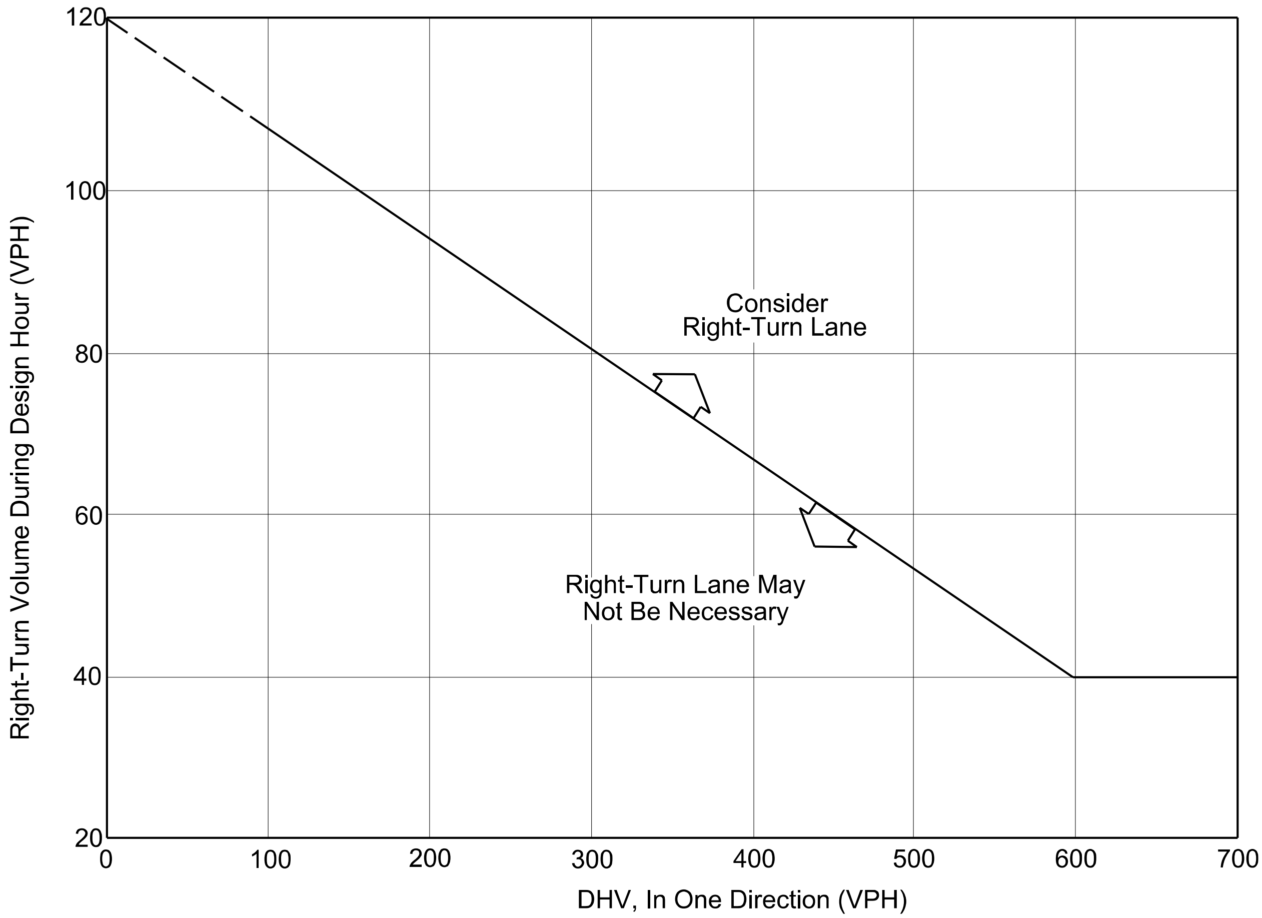
When the turning maneuver for left- and right-turning vehicles occurs in the through travel lanes, it disrupts the flow of through traffic. The use of auxiliary lanes may be warranted for at-grade intersections to improve the level of service and safety at the intersection.

9.5.1 Turn Lane Guidelines

9.5.1.1 Guidelines for Right-Turn Lanes

The use of right-turn lanes at intersections can significantly improve operations. Consider exclusive right-turn lanes:

- at the free-flowing leg of any unsignalized intersection on a two-lane urban or rural highway that satisfies the criteria in Figure 9.5-A;
- at the free-flowing leg of any unsignalized intersection on a high-speed (50 miles per hour or greater), four-lane urban or rural highway that satisfies the criteria in Figure 9.5-B;
- at the free-flowing leg of any unsignalized intersection on a six-lane urban or rural highway;
- at any intersection where a capacity analysis determines a right-turn lane is necessary to meet the overall level-of-service criteria;
- as a general rule, at any signalized intersection where the projected right-turning volume is greater than 300 vehicles per hour and where there are greater than 300 vehicles per hour per lane on the mainline (A traffic analysis will be required if the turning volumes are greater than 300 vehicles per hour.);
- for uniformity of intersection design along the highway if other intersections have right-turn lanes;
- at any intersection where the mainline is curved to the left and where the mainline curve requires superelevation;
- at railroad crossings where the railroad is parallel to the facility and is located close to the intersection and where a right-turn lane would be desirable to store queued vehicles avoiding interference with the movement of through traffic; or
- at any intersection where the crash experience, existing traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve) or engineering judgment indicates a significant conflict related to right-turning vehicles.



Note: *For highways with a design speed below 50 miles per hour with a DHV < 300 and where right turns > 40, an adjustment should be used. To read the vertical axis of the chart, subtract 20 from the actual number of right turns.*

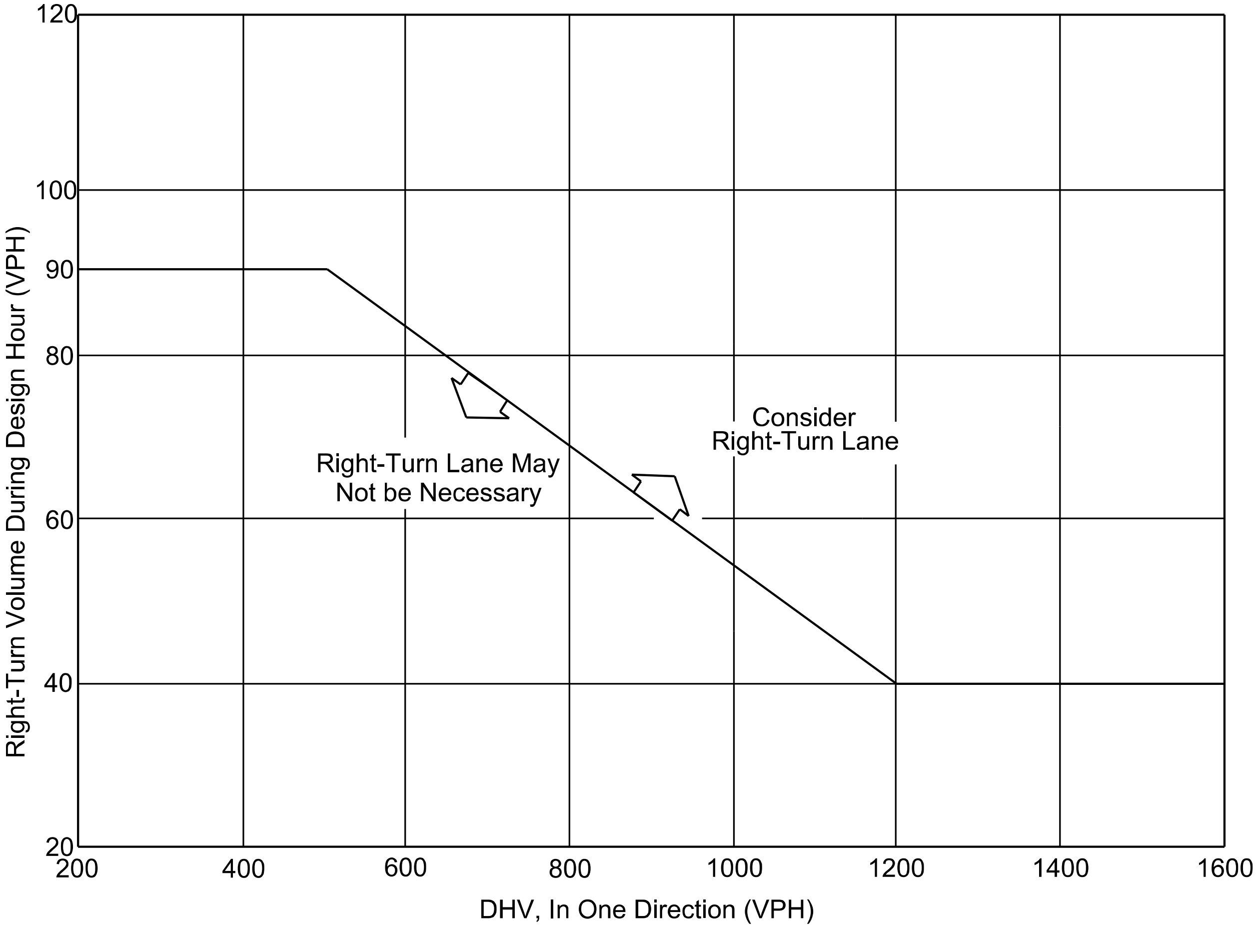
<u>Example</u>

<u>Given</u>:	Design Speed	=	35 miles per hour
	DHV	=	250 vehicles per hour
	Right Turns	=	100 vehicles per hour

<u>Problem</u>: Determine if a right-turn lane is necessary.

<u>Solution</u>: To read the vertical axis, use 100 – 20 = 80 vehicles per hour. The figure indicates that a right-turn lane is not necessary, unless other factors (e.g., high crash rate) indicate a lane is needed.

GUIDELINES FOR RIGHT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS
Figure 9.5-A



Note: Figure is only applicable on highways with a design speed of 50 miles per hour or greater.

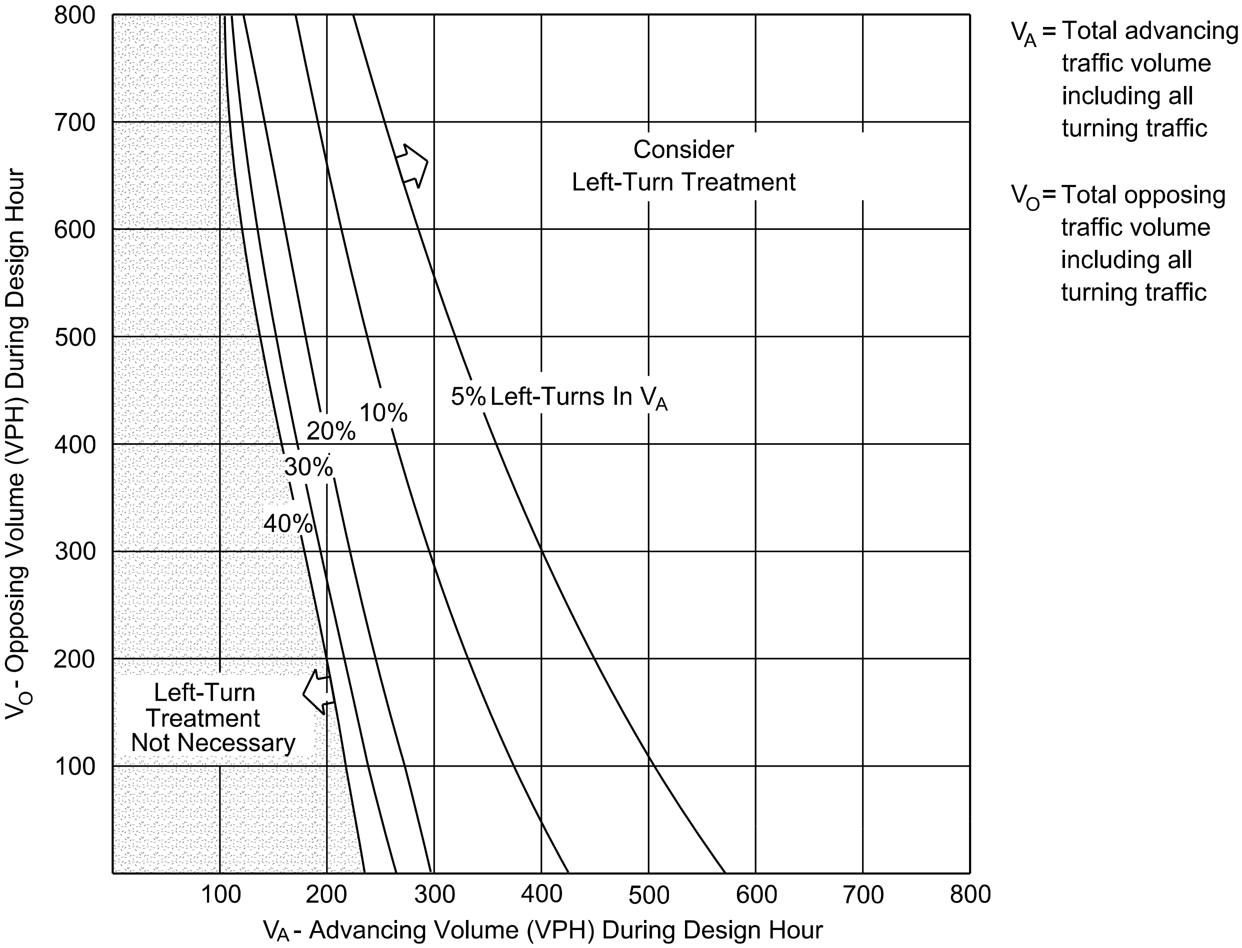
GUIDELINES FOR RIGHT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON FOUR-LANE HIGHWAYS
Figure 9.5-B

9.5.1.2 Guidelines for Left-Turn Lanes

The accommodation of left turns is often the critical factor in proper intersection design. Left-turn lanes can significantly improve both the level of service and intersection safety. Always use an exclusive left-turn lane at all intersections with public roads on divided urban and rural highways with a depressed median wide enough to accommodate a left-turn lane, regardless of traffic volumes. Consider using an exclusive left-turn lane for the following:

- at any unsignalized intersection on principal, high-speed rural highways with other arterials or collectors;
- at any unsignalized intersection on a two-lane urban or rural highway that satisfies the criteria in Figures 9.5-C, 9.5-D, 9.5-E, 9.5-F or 9.5-G;
- at any intersection where a capacity analysis determines a left-turn lane is necessary to meet the level-of-service criteria;
- at any signalized intersection where the left-turn volume is 300 vehicles per hour or more, conduct a traffic review to determine if dual left-turn lanes are required;
- as a general rule, at any intersection where the left-turning volume is 100 vehicles per hour (for a single turn lane) or 300 vehicles per hour (for a dual turn lane);
- at all entrances to major residential, commercial and industrial developments;
- at all median crossovers;
- for uniformity of intersection design along the highway if other intersections have left-turn lanes (i.e., to satisfy driver expectancy); or
- at any intersection where the crash experience, traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve) or engineering judgment indicates a significant conflict related to left-turning vehicles.

For additional information, see TRB Highway Research Record 211, *Volume Warrants for Left Turn Storage Lanes at Unsignalized Grade Intersections*.



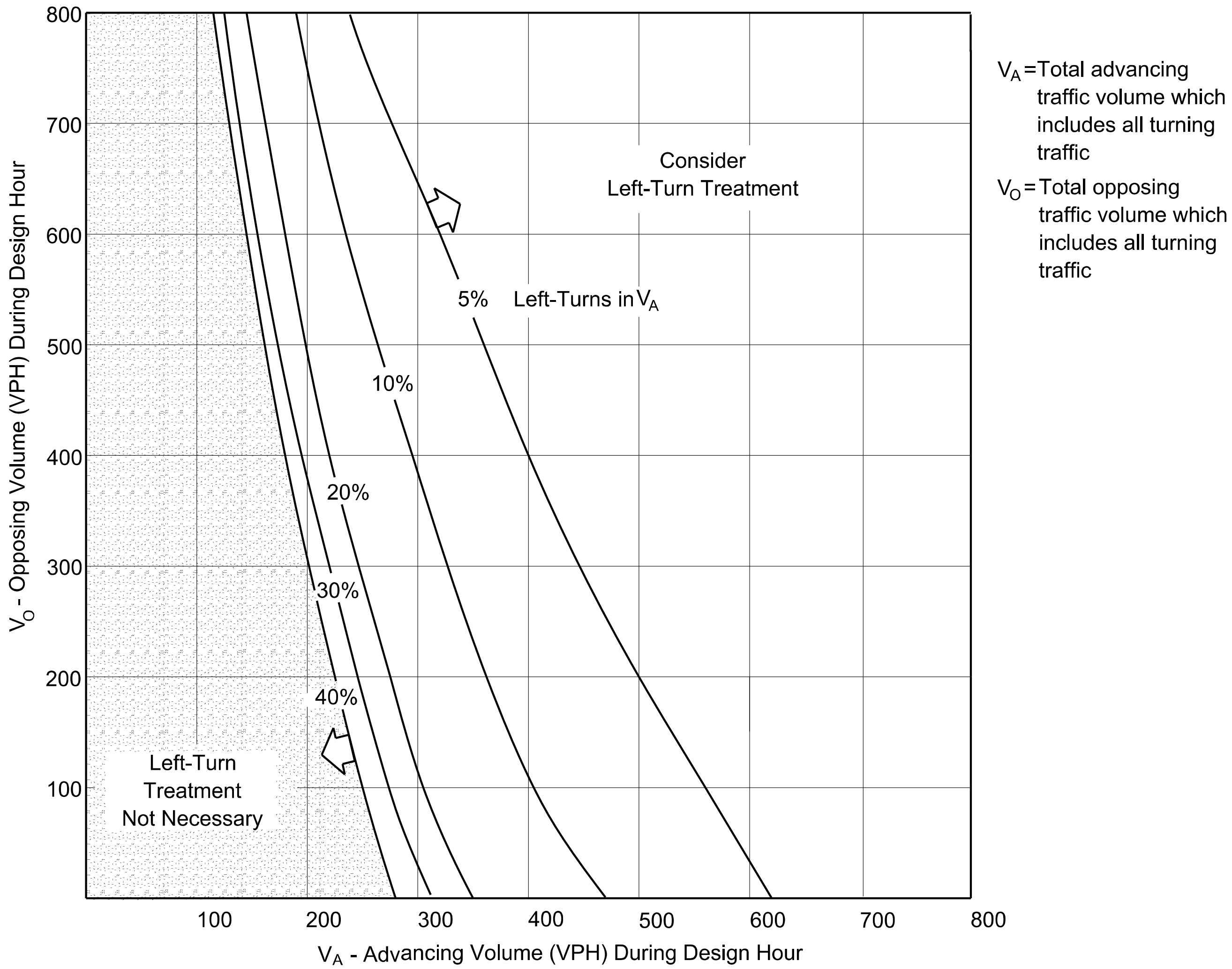
Instructions:

1. *The family of curves represents the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of 5, the designer should estimate where the curve lies.*

2. *Read V_A and V_O into the chart and locate the intersection of the two volumes.*

3. *Note the location of the point in #2 relative to the line in #1. If the point is to the right of the line, then a left-turn lane is warranted. If the point is to the left of the line, then a left-turn lane is not warranted based on traffic volumes.*

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS (60 mph)
Figure 9.5-C



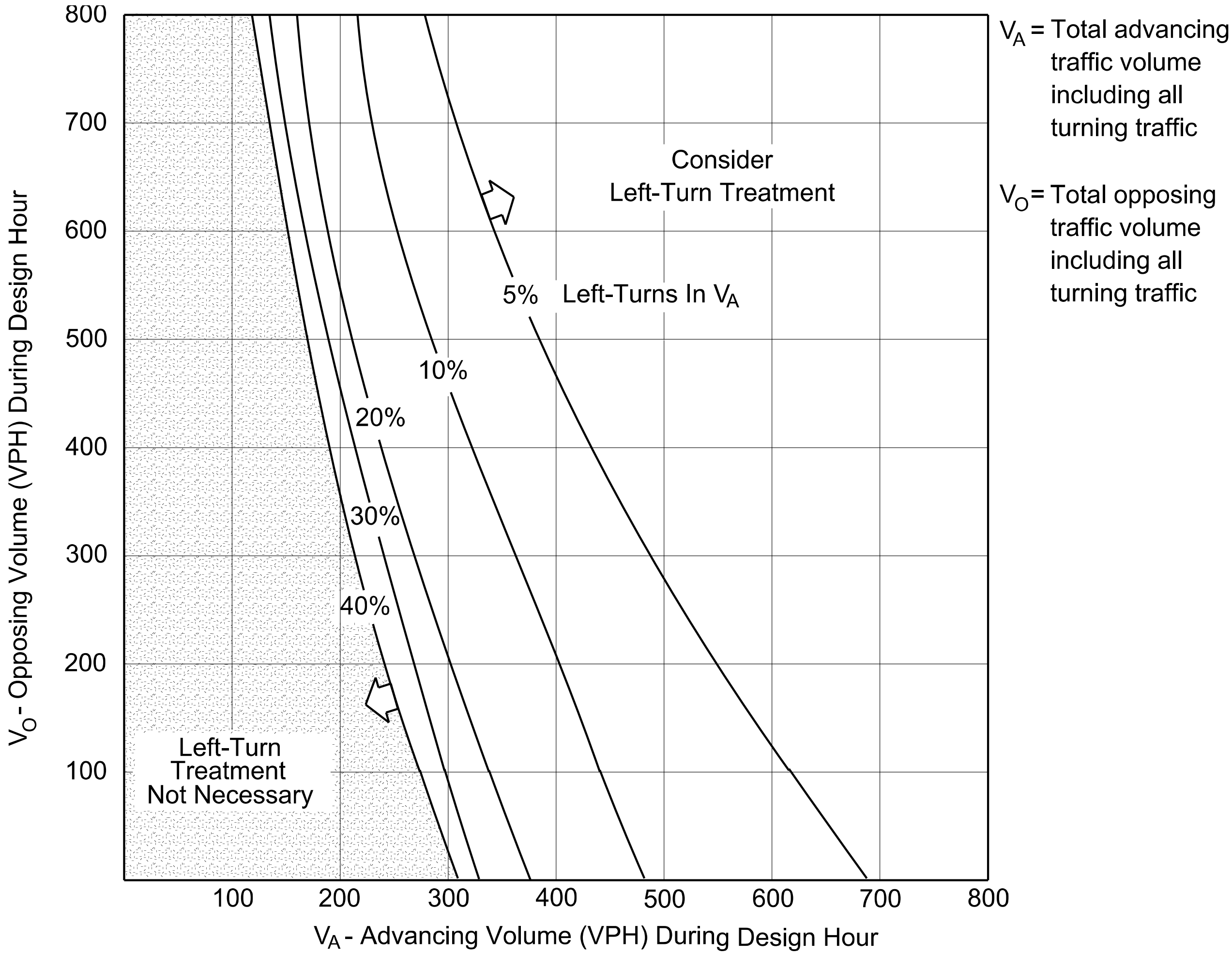
Instructions:

1. *The family of curves represents the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of 5, the designer should estimate where the curve lies.*

2. *Read V_A and V_O into the chart and locate the intersection of the two volumes.*

3. *Note the location of the point in #2 relative to the line in #1. If the point is to the right of the line, then a left-turn lane is warranted. If the point is to the left of the line, then a left-turn lane is not warranted based on traffic volumes.*

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS (55 mph)
Figure 9.5-D



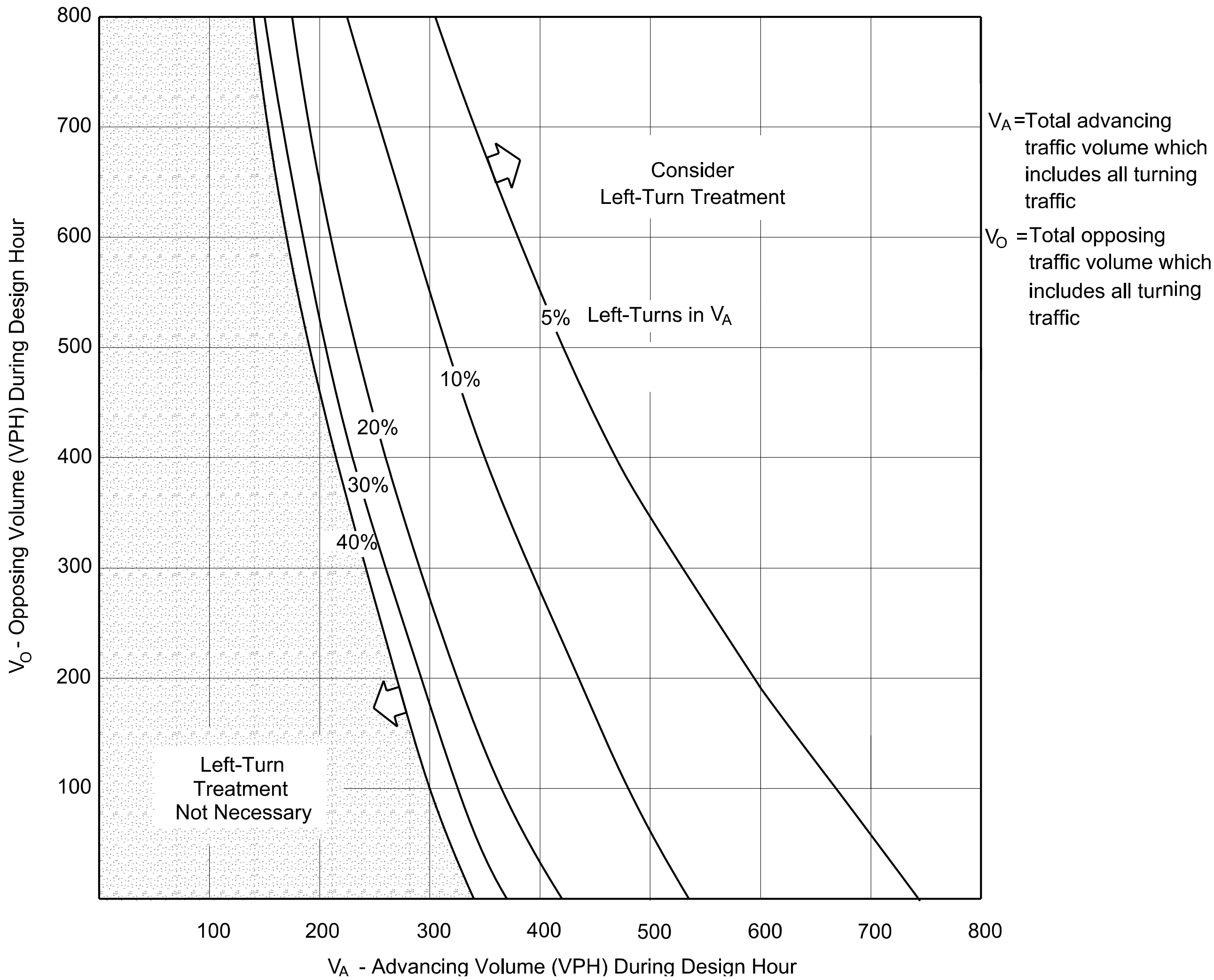
Instructions:

1. *The family of curves represents the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of 5, the designer should estimate where the curve lies.*

2. *Read V_A and V_O into the chart and locate the intersection of the two volumes.*

3. *Note the location of the point in #2 relative to the line in #1. If the point is to the right of the line, then a left-turn lane is warranted. If the point is to the left of the line, then a left-turn lane is not warranted based on traffic volumes.*

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS (50 mph)
Figure 9.5-E



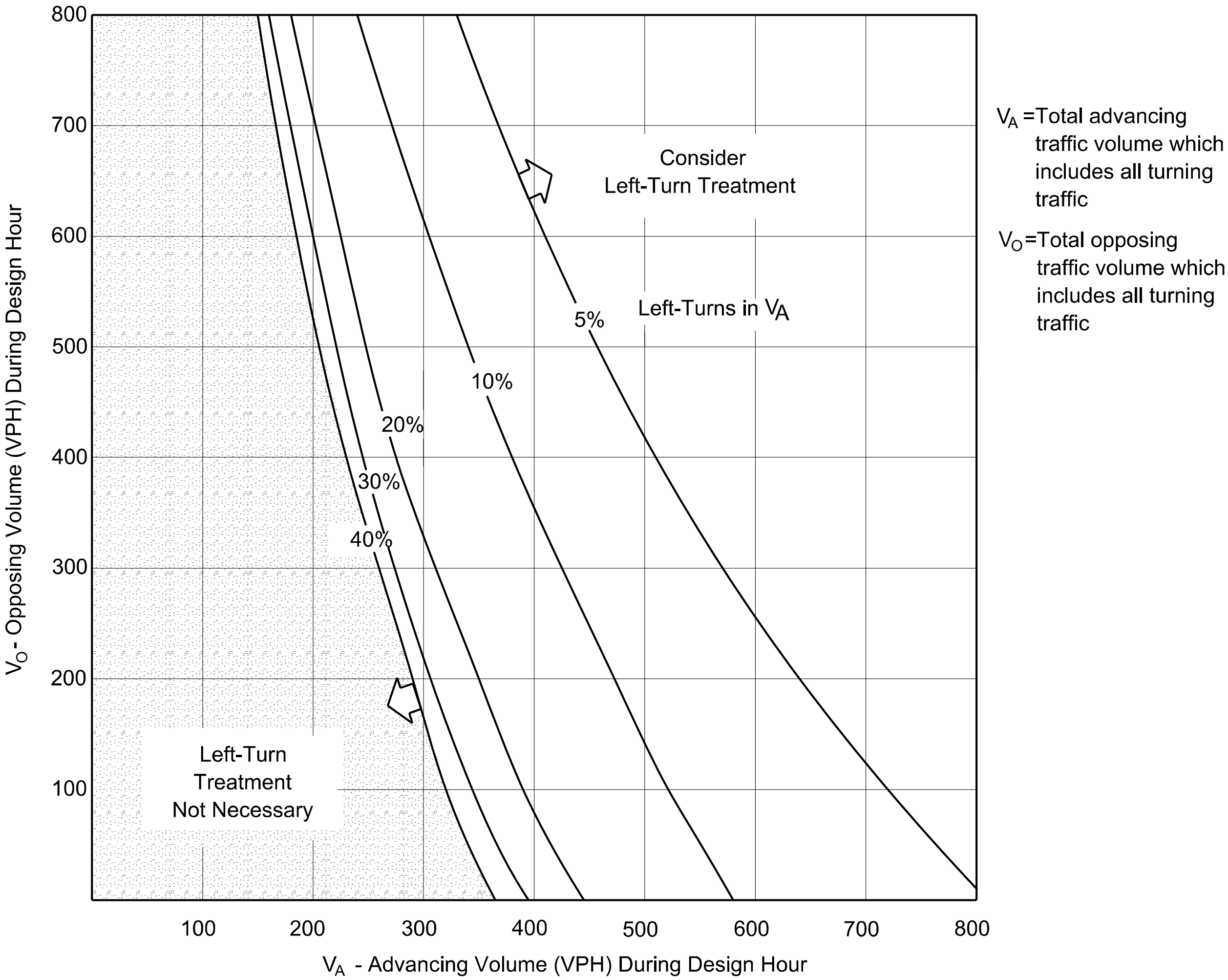
Instructions:

1. *The family of curves represents the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of 5, the designer should estimate where the curve lies.*

2. *Read V_A and V_O into the chart and locate the intersection of the two volumes.*

3. *Note the location of the point in #2 relative to the line in #1. If the point is to the right of the line, then a left-turn lane is warranted. If the point is to the left of the line, then a left-turn lane is not warranted based on traffic volumes.*

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS (45 mph)
Figure 9.5-F



Instructions:

1. *The family of curves represents the percent of left turns in the advancing volume (V_A). The designer should locate the curve for the actual percentage of left turns. When this is not an even increment of 5, the designer should estimate where the curve lies.*

2. *Read V_A and V_O into the chart and locate the intersection of the two volumes.*

3. *Note the location of the point in #2 relative to the line in #1. If the point is to the right of the line, then a left-turn lane is warranted. If the point is to the left of the line, then a left-turn lane is not warranted based on traffic volumes.*

VOLUME GUIDELINES FOR LEFT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON TWO-LANE HIGHWAYS (40 mph)
Figure 9.5-G

9.5.2 Design of Turn Lanes

9.5.2.1 Widths

The following will apply to auxiliary turn lane widths:

1. Lane Widths. Typically, the width of any turn lanes at an intersection will be the same as that of the adjacent through lane. In restricted areas, it may be justified to provide a narrower width.

2. Shoulder. The designer should meet the following for shoulders adjacent to auxiliary lanes:

    a. On Facilities without Curbs. The shoulder width adjacent to the auxiliary lane should be the same as the normal shoulder width for the roadway. At a minimum, the width may be 6 feet, assuming the roadway has a shoulder width equal to or greater than 6 feet.

    b. On Facilities with Curbs. The offset between the auxiliary lane and face of curb should be the same as that for the normal roadway section, typically the gutter width.

3. Cross Slope. The cross slope for an auxiliary lane will depend on the number of lanes and cross slope of the adjacent traveled way. See Section 7.2.6.1 for information on auxiliary lane cross slopes.

9.5.2.2 Turn Lane Lengths

The functional length of turn lanes is determined by the perception-reaction time (L_1), taper length (L_2), deceleration length (L_3) and storage length (L_4); see Figure 9.5-H. Desirably, under unconstrained conditions, the physical length of a right- or left-turn turn lane length at an intersection should allow for both safe vehicular deceleration (L_2 + L_3) and storage of turning vehicles (L_4) outside of the through lanes. Under constrained conditions, the minimum length of the turn lane will be determined by a combination of its taper length (L_2) and storage length (L_4) (i.e., deceleration occurs in the through travel lanes). The following will apply:

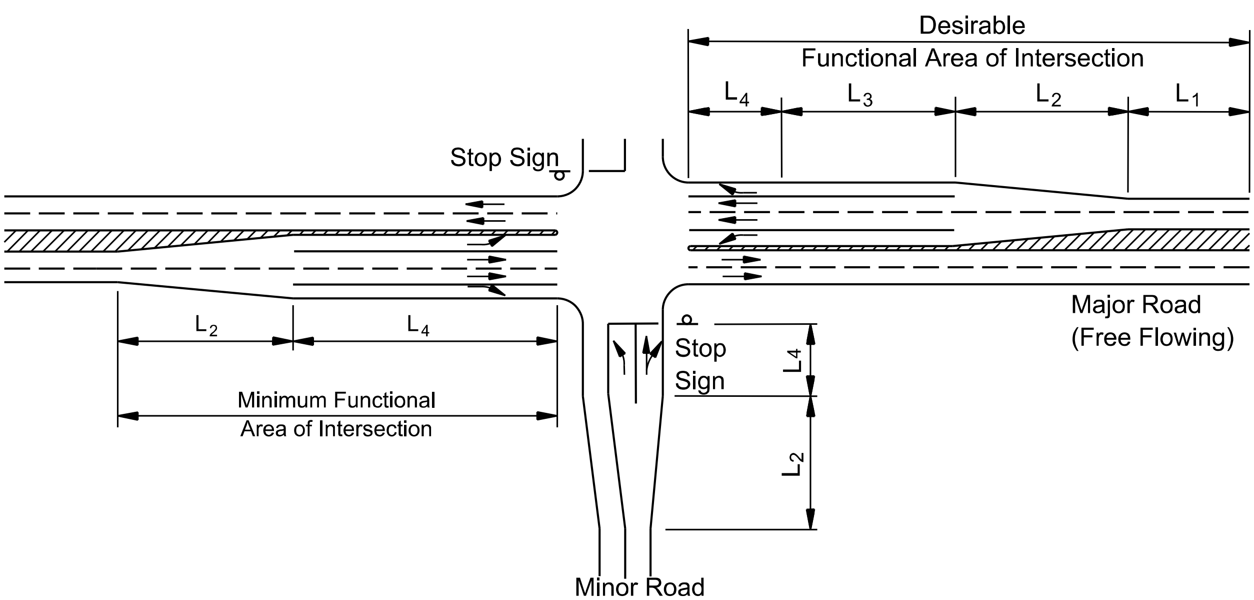
1. Tapers. The entrance taper (L_2) into the turn lane may be either a straight or a reverse curve taper. Typically, SCDOT uses reverse curves where the turn lane taper is painted. For other situations, straight tapers are commonly used (e.g., curb and gutter tapers). Figure 9.5-I provides the recommended taper distances for reverse-curve and straight tapers. Where the highway is on a curved alignment, the taper of the turn lane should be more pronounced than usual to ensure that the through motorists are not inadvertently directed into the turn lane. This is accomplished by shortening the taper length.

2. Deceleration. Desirably, all vehicular deceleration will occur within the taper and full width of the turn lane (L_2 + L_3); however, this is often impractical. Consequently, some or all deceleration may occur prior to the beginning of the taper. Figure 9.5-J provides desirable full deceleration lengths.

3. Right-Turn Lanes. Figure 9.5-K provides the minimum storage lengths (L_4) for right-turn lanes. The designer should use the traffic analysis to determine the actual storage length required.

4. Left-Turn Lanes. Figure 9.5-L provides the minimum storage lengths (L_4) for left-turn lanes. The designer should use the traffic analysis to determine the actual storage length required.

5. Queue Lengths. The designer should determine the stopped queue length for the through travel lane to ensure the queue does not block vehicles desiring to move into the turn lane. The turn lane should be extended beyond this point.



L_1 = Distance traveled during perception-reaction time, feet
L_2 = Taper distance to begin deceleration and complete lateral movement, feet
L_3 = Distance traveled to complete deceleration to a stop, feet
L_4 = Storage length, feet

Note: The schematic of the major road (free flowing) also applies to all legs of a signalized intersection.

FUNCTIONAL LENGTHS OF AUXILIARY TURN LANES
Figure 9.5-H

Reverse Curve Taper (L_2)					Straight Taper (L_2)	
Design Speed (mph)	**Radius (ft)**	**Auxiliary Lane Widths (ft)**			**Design Speed (mph)**	**Straight Taper Rate**
		W=10 ft	**W=11 ft**	**W=12 ft**		
V ≤ 30	300	109	115	120	V ≤ 30	8:1
31 - 40	480	138	145	152	31 – 40	12:1
41 - 50	670	163	171	179	V ≥ 41	15:1
V ≥ 51	840	183	192	201		

Notes:

1. *Create taper equivalent reverse curves.*
2. *Taper distance is approximately based on tangent alignment.*
3. *Based on the following formula:*

$$L = \sqrt{W(4R - W)}$$

Where:

L = *Length of reverse curve taper, feet*
W = *Width of auxiliary lane, feet*
R = *Radius, feet*

Notes:

1. *Where through road is on a curve, develop a uniform offset taper from the curved mainline.*

TYPICAL AUXILIARY LANES TAPER LENGTHS
Figure 9.5-I

Design Speed (mph)	Distance (ft)
30	160
40	275
50	425
60	605
70	820

Notes:

1. *The above full deceleration lengths are L_2 + L_3. The initial speed is equal to the average running speed.*
2. *Assumes a turning vehicle has cleared the through lane when it has moved laterally approximately 9 feet so that a following through vehicle can pass without encroaching upon the adjacent travel lane.*
3. *The speed differential between the turning vehicle and following through vehicle is 10 miles per hour when the turning vehicle clears the through travel lane.*
4. *The turn vehicle decelerates at a rate of 5.8 ft/s² while moving from the through lane into the turn lane and then at rate of 6.5 ft/s² after completing the lateral shift into the turn lane.*

DESIRABLE DECELERATION LENGTHS
Figure 9.5-J

Turning Volume (vph)	Percent of Trucks in Turning Volume				
	0 to 10%	**20%**	**40%**	**60%**	**100%**
50	Use Minimum Length of 100 ft				
100					125 ft
150		125 ft	175 ft	175 ft	175 ft
200	150 ft	175 ft	225 ft	225 ft	250 ft
250	200 ft	225 ft	275 ft	275 ft	325 ft
300	250 ft	275 ft	325 ft	350 ft	400 ft
350	300 ft	325 ft	375 ft	425 ft	475 ft
400	350 ft	375 ft	425 ft	500 ft	550 ft

Note: The traffic designer should review the design to determine if longer turn lane lengths are required.

GUIDELINES FOR RIGHT-TURN LANE LENGTHS (L_4)
Figure 9.5-K

Turning Volume (vph)	Percent of Trucks in Turning Volume				
	0 to 10%	**20%**	**40%**	**60%**	**100%**
50	In Urban Areas, Use Minimum Length of 150 ft In Rural Areas, Use Minimum Length of 200 ft				
100					
150			175 ft	175 ft	175 ft
200		175 ft	225 ft	225 ft	250 ft
250	200 ft	225 ft	275 ft	275 ft	325 ft
300	250 ft	275 ft	325 ft	350 ft	400 ft
350	300 ft	325 ft	375 ft	425 ft	475 ft
400	350 ft	375 ft	425 ft	500 ft	550 ft

Notes:

1. *Consider providing dual-turn lanes if the turning volumes are greater than 300 vehicles per hour. A traffic analysis will be required if the turning volumes are greater than 300 vehicles per hour.*

2. *The traffic designer should review the design to determine if longer turn lane lengths are required.*

GUIDELINES FOR LEFT-TURN LANE LENGTHS (L_4)
Figure 9.5-L

9.5.3 Typical Turn Lane Treatments

Various turn lane designs are illustrated in Figures 9.5-M through 9.5-O. In addition, the designer should consider the following:

1. Two-Lane Facilities. If a left-turn lane is required on a two-lane highway, it should desirably be designed as a fully channelized left-turn lane. A typical channelized left-turn lane is illustrated in Figure 9.5-M. Generally, left-turn deceleration and storage bays will be designed symmetrically about the highway centerline.

2. Divided Facilities. Figure 9.5-N illustrates typical treatments for left- and right-turn lanes on divided facilities. Left-turn lanes will generally be the parallel design.

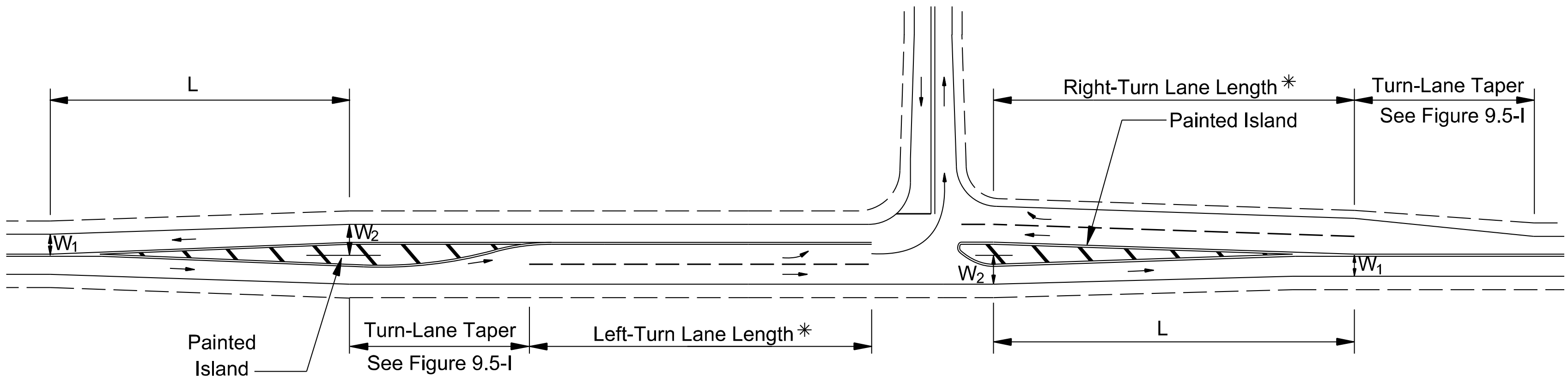
3. Offset Left-Turn Lanes. On medians wider than 17 feet, it is desirable to align the left-turn lane so that it will reduce the width of the median nose to 1 to 6 feet. This alignment will place the vehicle waiting to make the turn as far to the left as practical, maximize the offset between the opposing left-turn lanes, and provide improved visibility to the opposing through traffic. The advantages of offsetting the left-turn lanes are:

    - better visibility of opposing through traffic;
    - decreased probability of a conflict between opposing left-turn movements within the intersection; and
    - more left-turn vehicles can be served in a given time period, especially at signalized intersections.

    Offset designs may be either the parallel or taper design; see Figure 9.5-O. The parallel design may be used at signalized and unsignalized intersections. However, the taper design is primarily only used at signalized intersections. Offset left-turn lanes should be separated from the adjacent through traveled way by painted or raised channelization.

4. Offset Right-Turn Lanes. A potential problem in installing right-turn lanes at intersections is that vehicles in the right-turn lane on the major road may block the minor-road drivers' views of traffic approaching on the major road. This can lead to crashes between vehicles turning left, turning right or crossing from the minor road and through vehicles on the major road. To reduce the potential for crashes of this type, the right-turn lane can be offset by moving it laterally so that vehicles in the right-turn lane no longer obstruct the view of the minor road driver.

    Installation of offset right-turn lanes increases the overall width of the intersection, which may cause potential problems for pedestrians crossing the intersection. A possible solution to this problem would be to provide a pedestrian refuge island between the offset right-turn lane and through lanes. However, it is not advisable to use raised islands on high speed approaches.



Taper Length (L) = WS (S > 40), or L = WS²/60 (S ≤ 40)

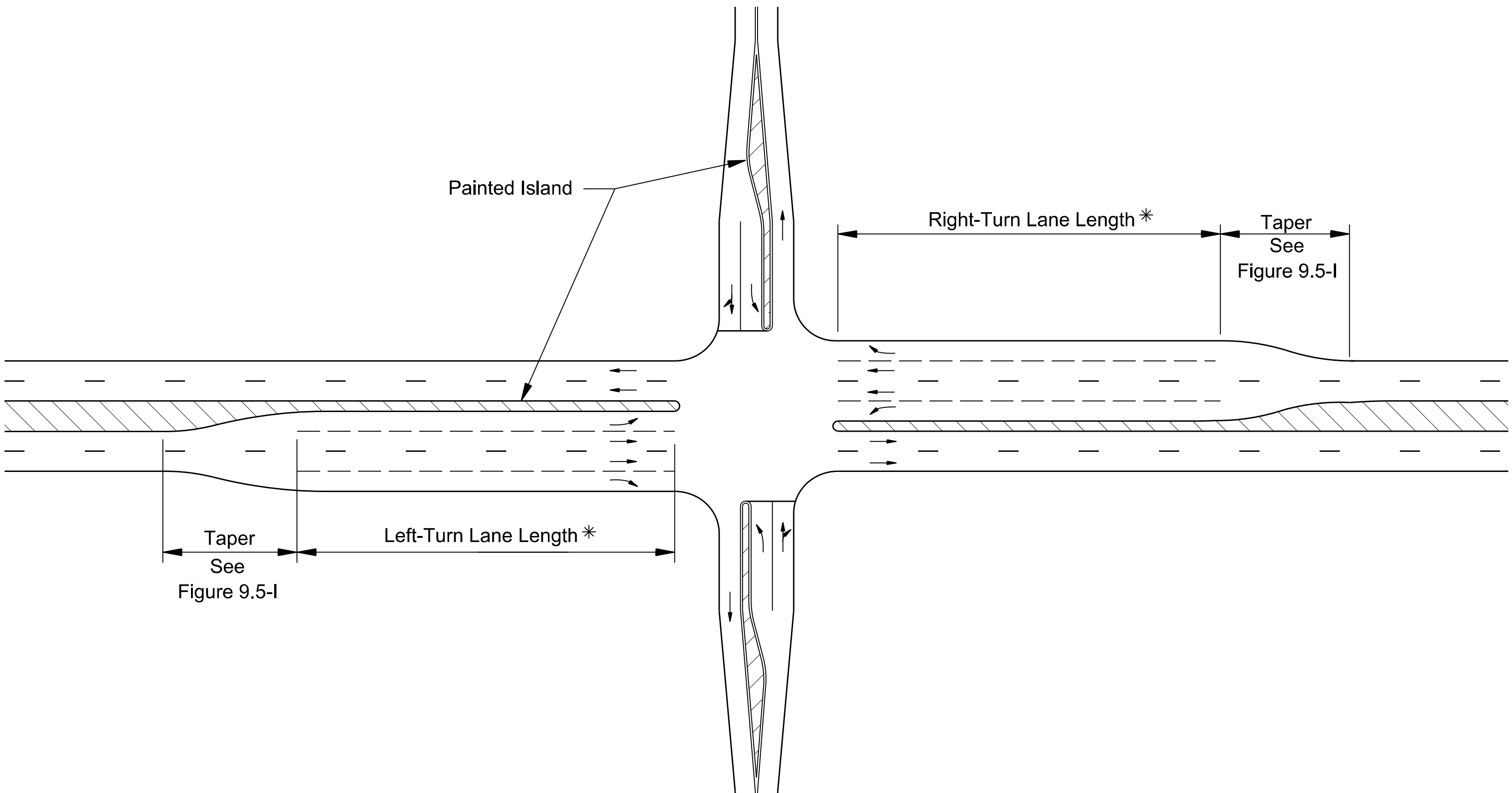
Where:

L = Taper Length, feet
W = Transition Width ($W_2 – W_1$), feet
S = Design Speed, miles per hour

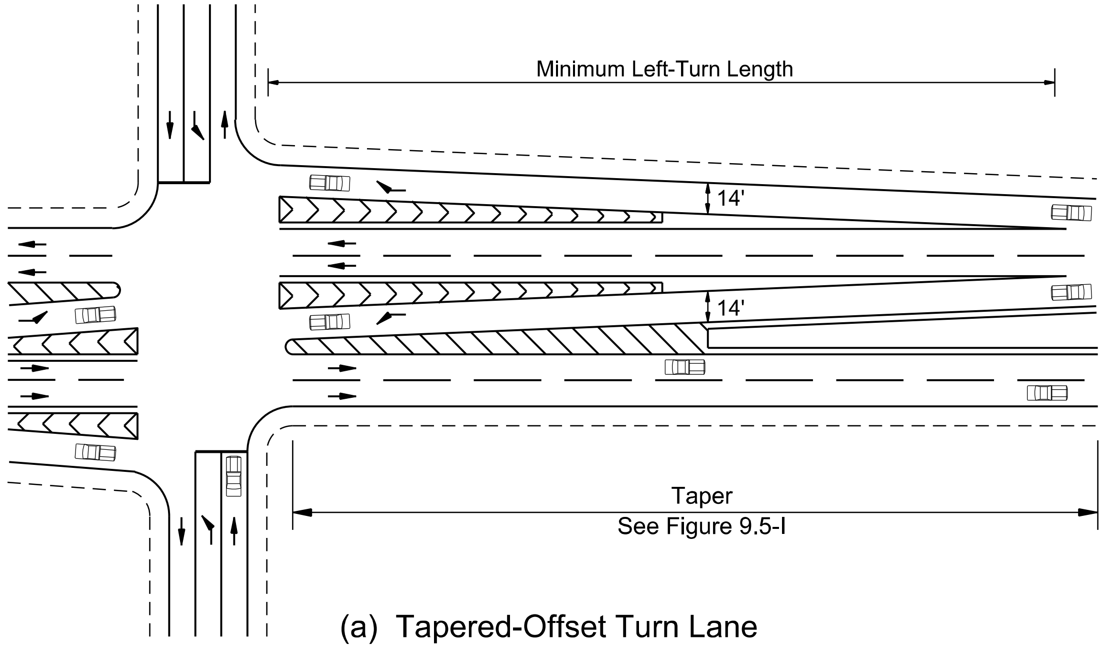
**See Section 9.5.2.2 for minimum turn lane lengths.*

TURN LANES FOR TWO-LANE HIGHWAYS
Figure 9.5-M

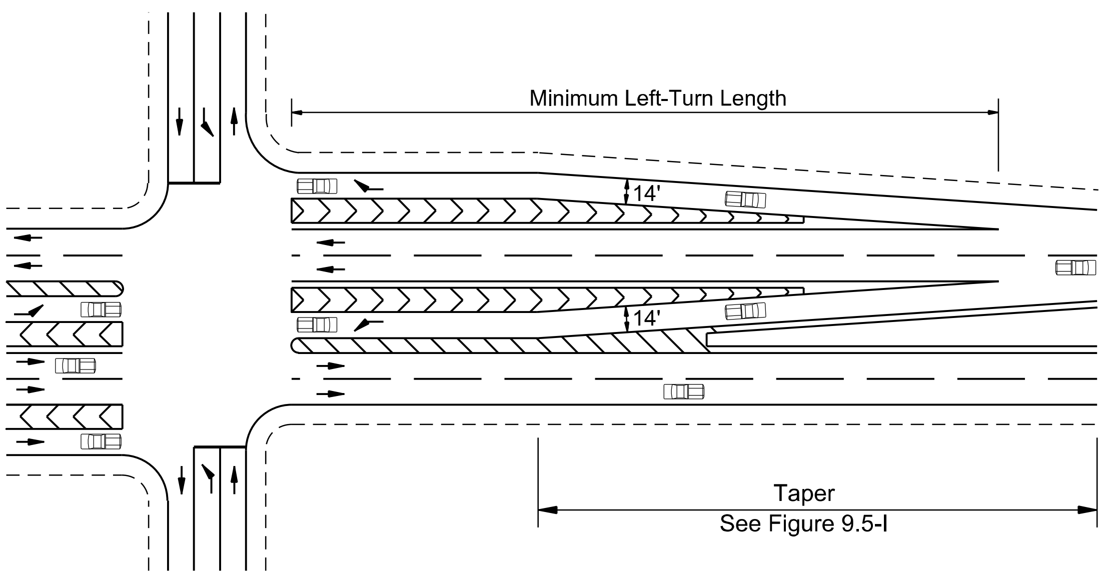
TURN LANES FOR FOUR-LANE HIGHWAYS
Figure 9.5-N



**See Section 9.5.2.2 for minimum turn-lane lengths.*



(a) Tapered-Offset Turn Lane



(b) Parallel-Offset Turn Lane

See Section 9.5.2.2 for minimum turn-lane lengths.

OFFSET LEFT-TURN LANES
Figure 9.5-O

9.5.4 Dual-Turn Lanes

9.5.4.1 Guidelines

At signalized intersections with high-turning volumes, the designer may consider providing dual left- and/or right-turn lanes. However, multiple turn lanes may cause problems with right of way, lane alignment, crossing pedestrians and lane confusion for approaching drivers. Consider using dual right- and left-turn lanes where:

- based on the traffic analysis, the necessary time for a protected left-turn phase becomes unattainable to meet the level-of-service criteria (average delay per vehicle); and/or

- there is insufficient space to provide the calculated length of a single-turn lane because of site restrictions (e.g., closely spaced intersections).

Dual right-turn lanes do not work as well as dual left-turn lanes because of the more restrictive space available for two-abreast right turns. If practical, the designer should find an alternative means to accommodate the high number of right-turning vehicles. For example, a turning roadway may accomplish this purpose.

9.5.4.2 Design

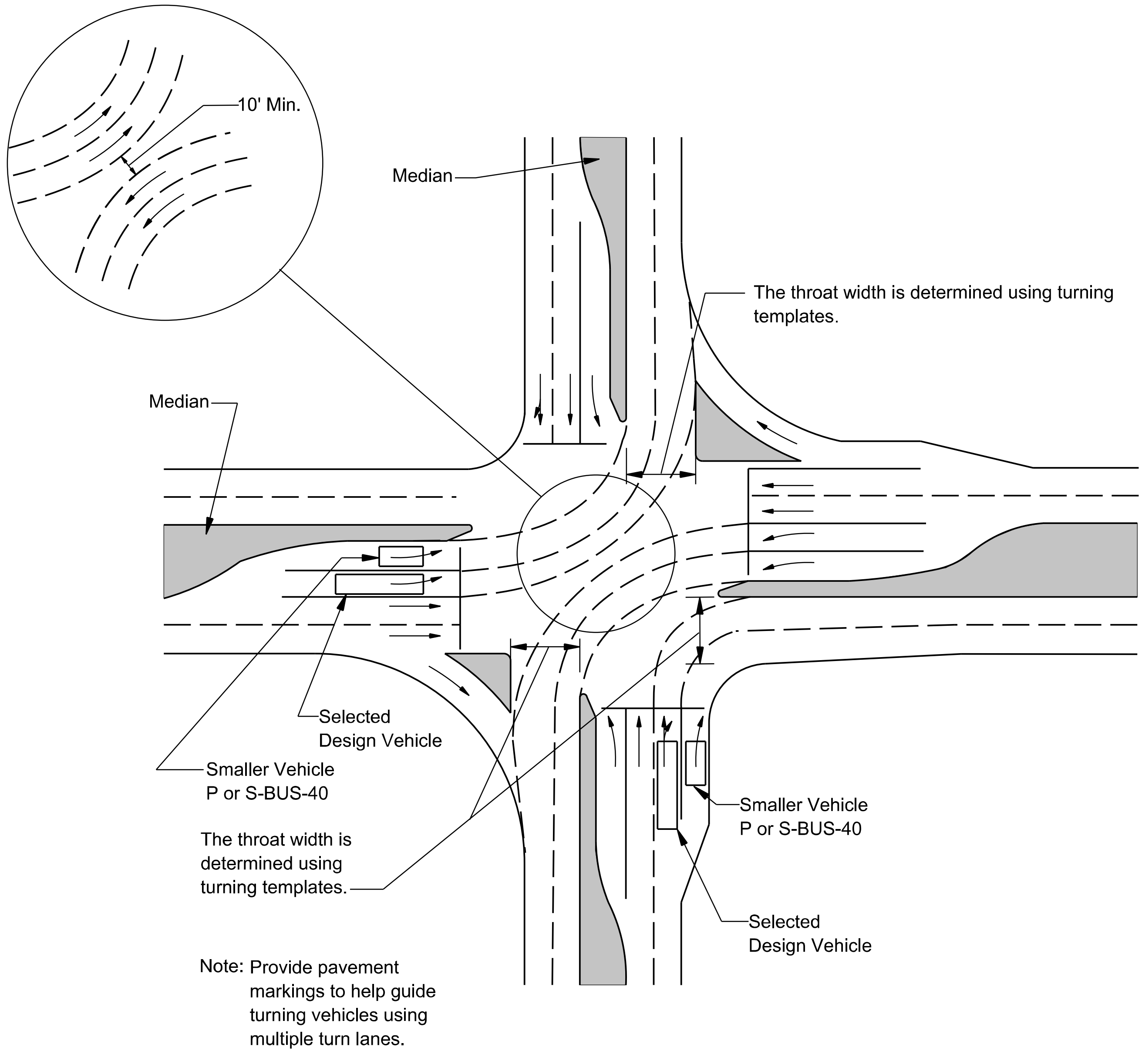
A dual-turn lane (both lanes exclusive) can potentially discharge approximately 1.9 times the number of cars that will discharge from a single exclusive turn lane. However, to work properly, the designer must carefully consider several design elements. Figure 9.5-P presents both dual right- and left-turn lanes to illustrate the more important design elements. The designer should consider the following:

1. Turn Lane Lengths. See Section 9.5.2.2.

2. Turning Templates. Using the applicable turning templates, check all intersection design elements. The designer should assume that the larger selected design vehicle would turn from the lane with the widest turn. Typically, the S-BUS-40 design vehicle will turn side-by-side with the selected design vehicle. See Figure 9.5-P for the design vehicle placement.

3. Throat Width. Because of the off-tracking characteristics of turning vehicles, the normal width of two travel lanes may be inadequate to properly receive two vehicles turning abreast. Therefore, the receiving throat width may need to be widened. This should be checked using turning templates. For 90-degree intersections, the designer can expect that the throat width for dual-turn lanes will be approximately 30 feet to 36 feet. When determining the available throat width, the designer can assume that the paved shoulder, if present, will be used to accommodate two-abreast turns.

4. Widening Approaching Through Lanes. If a 30-foot to 36-foot throat width is provided to receive dual-turn lanes, the designer should also consider how this will affect the traffic approaching from the other side. The designer should also ensure that the through lanes line up relatively well to ensure a smooth flow of traffic through the intersection.



DUAL-TURN LANES
Figure 9.5-P

5. Median Widths. It is desirable to have a median width of at least 28 feet for dual left-turn lanes.

6. Pavement Markings. Pavement markings can effectively guide two lanes of vehicles turning abreast. See the *MUTCD* for applicable guidelines on the selection and placement of any special pavement markings.

7. Opposing Left-Turn Traffic. It is desirable that opposing left turns occur simultaneously; therefore, the designer should ensure that there is sufficient space for all turning movements. The separation between turn lanes should be 10 feet; see Figure 9.5-P. If space is unavailable, it will be necessary to alter the signal phasing to allow the two directions of turning traffic to move through the intersection on separate phases.

8. Length of Receiving Lanes. Dual left-turn lanes require two receiving lanes. If two lanes are not required beyond the intersection, continue the second lane for at least 1000 feet, excluding the lane drop taper, before dropping the extra lane.

9.5.5 Acceleration Lanes

On multilane facilities, acceleration lanes may be considered near industrial parks or other major traffic generators. The acceleration design lengths can be determined by reviewing the acceleration distances in Chapter 10 “Interchanges” for ramps and the AASHTO *A Policy on the Geometric Design of Highways and Streets*.

9.6 CHANNELIZATION

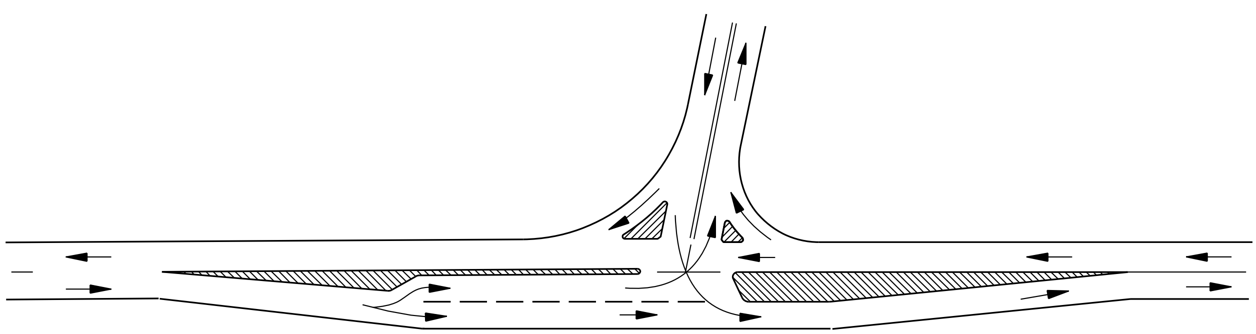
9.6.1 Functional Types

Islands can be grouped into the following functional classes. Most islands serve at least two of these functions:

1. Divisional Islands. Divisional islands segregate opposing traffic flows, alert the driver to the crossroad ahead and regulate traffic through the intersection. These islands are often introduced at intersections on undivided highways and are particularly advantageous in controlling left turns at skewed intersections. This prevents conflicts with stored vehicles stopped at the stop bar and improves the intersection sight triangles. Figure 9.6-A illustrates an intersection with divisional islands.

2. Channelization Islands. Channelization islands control and direct traffic movements and guide the driver into the proper channel. Corner islands are one type of channelization island. Corner islands at or near crosswalks channelize right-turning vehicles and aid or protect pedestrians crossing a wide roadway.

3. Refuge Islands. Refuge islands at or near crosswalks or bicycle paths aid or protect pedestrians and bicyclists crossing a wide roadway. These islands may be required for pedestrians at intersections where complex signal phasing is used. The island width must be sufficient to meet the anticipated storage needs (pedestrians and bicyclists) and/or accessibility needs (e.g., wheelchairs) for refuge purposes.



DIVISIONAL ISLANDS
Figure 9.6-A

9.6.2 Selection of Channelization

Channelization may consist of some combination of flush or raised, concrete or grass, and may be triangular or elongated. Selection of an appropriate type of channelization will be determined on a case-by-case evaluation based on traffic characteristics, cost considerations and maintenance needs. The following sections offer guidance for channelization.

9.6.2.1 Flush Channelization

Flush channelization is appropriate:

- at locations requiring delineation of vehicular paths, such as at major route turns or intersections with unusual geometry;
- on high-speed rural highways to delineate separate turning lanes;
- in restricted locations where vehicular path definition is desired, but space for larger, raised channelization is not available; and/or
- to separate opposing traffic streams on low-speed streets.

9.6.2.2 Raised Channelization

Raised channelization emphasizes the location of the movement to be completed. Raised channelization is appropriate:

- at locations requiring positive delineation of vehicular paths, such as at major route turns or intersections with unusual geometry;
- where a primary function of an island is to provide a pedestrian refuge;
- where a primary or secondary island function is the location of traffic signals, signs or other traffic control features;
- where the channelization is intended to prohibit or prevent traffic movements; and/or
- on low- to moderate-speed highways where the primary function is to separate high volumes of opposing traffic movements.

Where a raised channelization is selected, a sloping concrete curb will typically be used. The need for lighting the intersection will be determined on a case-by-case basis.

9.6.3 <u>Size</u>

Corner islands and divisional islands should be large enough to command the driver's attention. Shapes and sizes will vary from one intersection to another. The following will apply:

1. <u>Corner Islands</u>. The recommended minimum size is 50 square feet (urban) and 100 square feet (rural). Desirably, all triangular islands will be at least 100 square feet, if practical. Islands used for pedestrian refuge should be at least 150 square feet to allow for the construction of curb ramps or channels for the disabled.

2. <u>Divisional Islands</u>. The recommended minimum width is 4 feet and desirably 10 feet wide. Divisional islands should not be less than 20 to 25 feet long.

9.6.4 Offset to Through Lanes

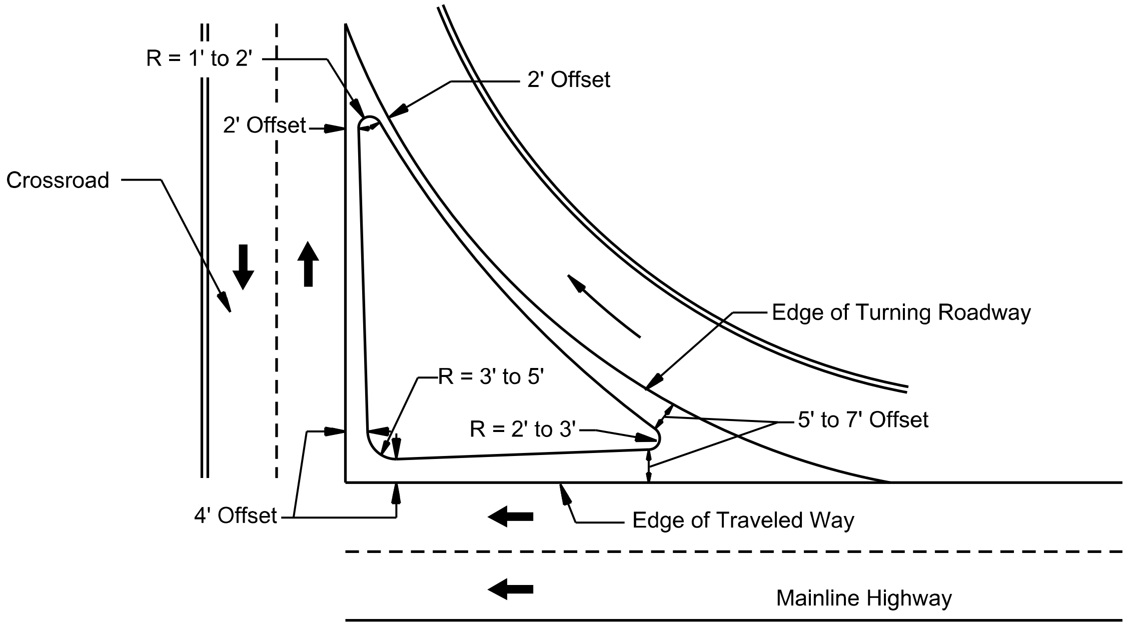
In urban areas on approaching roadways with curb offsets, offset the raised channelization at least 2 feet from the travel lane (Figure 9.6-B(a)). Where shoulders are present, offset raised channelization a distance equal to the shoulder width (Figure 9.6-B(b)). In rural areas and where separate turning lanes are used, offset the island from the turning lane by at least 2 feet. If there are no turning lanes, the island should be offset a distance equal to the shoulder width.

9.6.5 Delineation

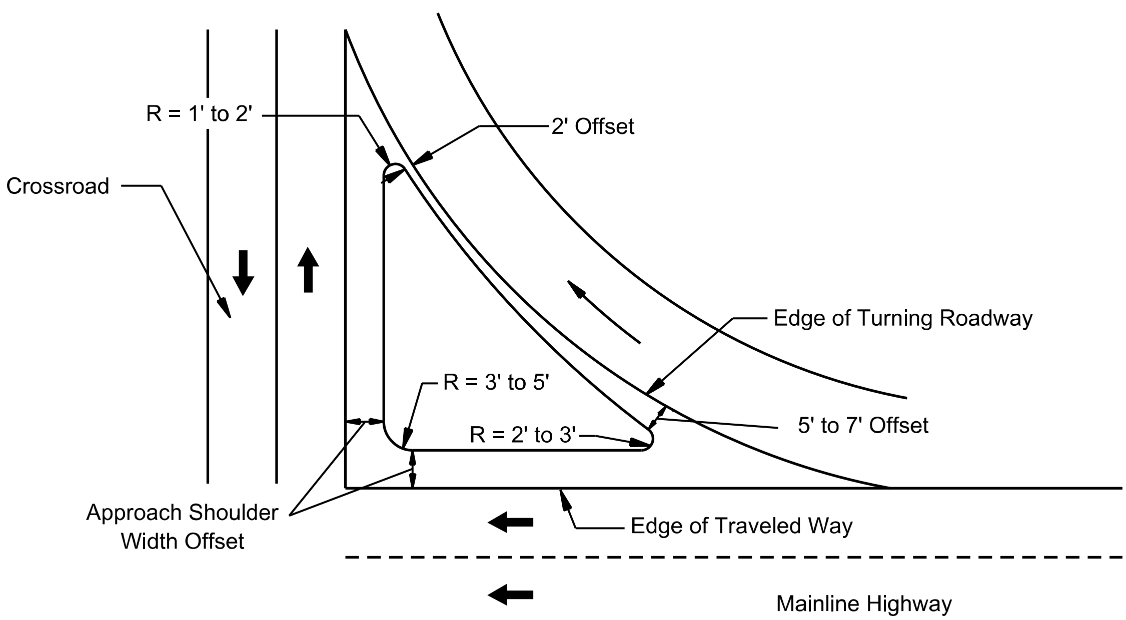
Pavement markings are used in advance of and around the raised channelization to warn the driver. These traffic control devices are especially important at the approach to divisional islands for the direction of approaching traffic. For guidance on pavement markings around divisional islands; see the *MUTCD*.

9.6.6 Concrete Island in Flush Median

Concrete islands may be used at locations requiring positive delineation of vehicular paths, such as at major route turns or intersections with unusual geometry or where the channelization is intended to prohibit or prevent traffic movements. Figure 9.6-C depicts the use of concrete divisional islands for left-turn lanes at intersections. The left-turn lane storage requirements and the design vehicle turning path will determine the length and location of the concrete divisional island.

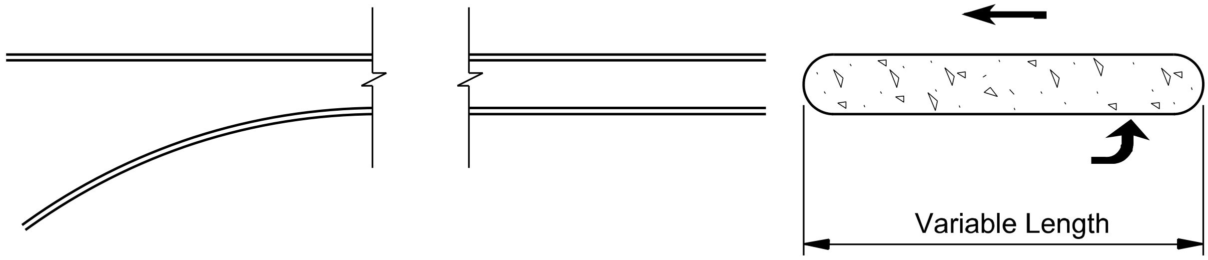


(a) Street with Curb and Gutter



(b) Road with Shoulders

CORNER ISLANDS
Figure 9.6-B



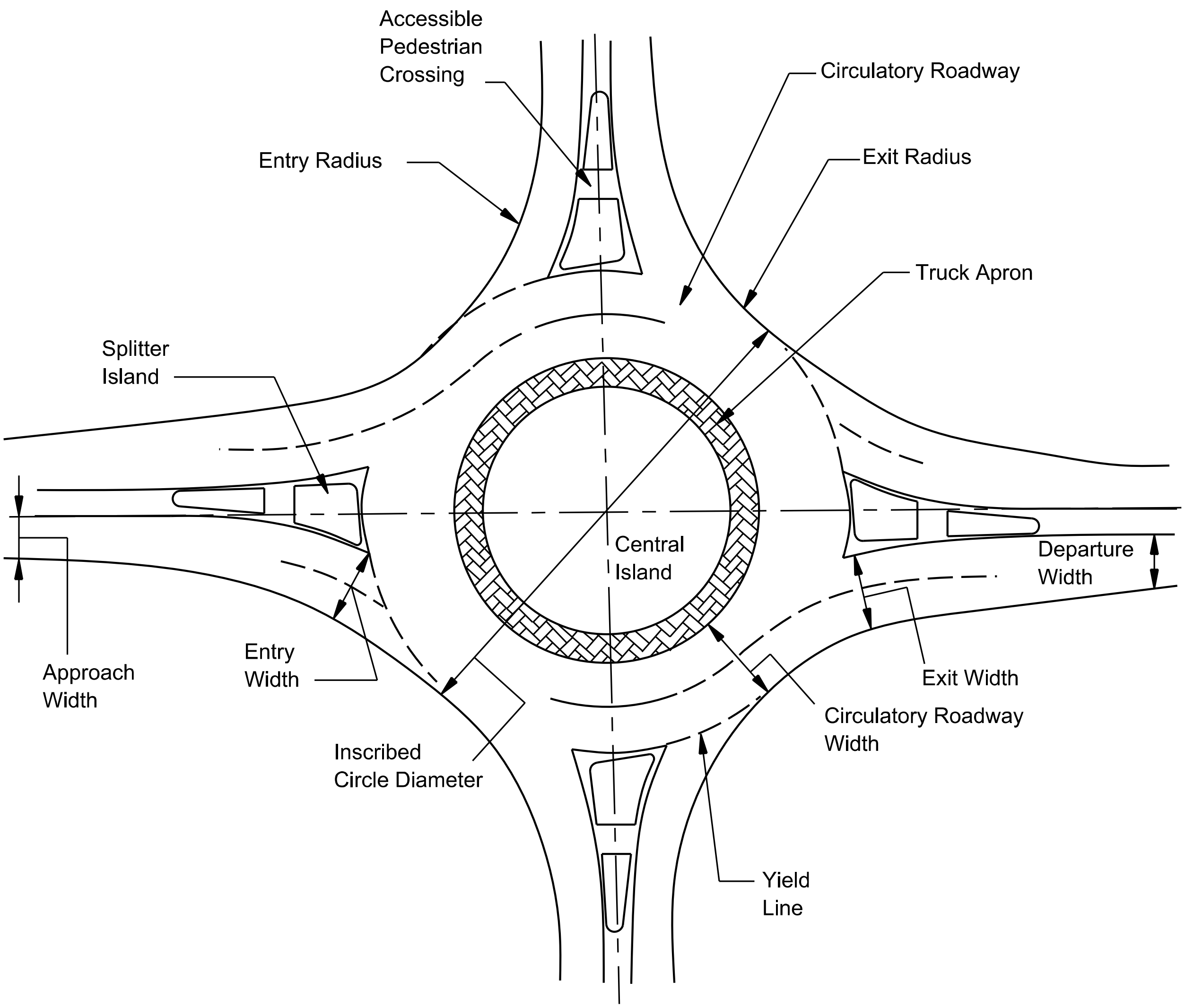
CONCRETE DIVISIONAL ISLANDS
(Location with Left-Turn Lane)
Figure 9.6-C

SPACER PAGE

9.7 ROUNDABOUTS

9.7.1 General

Roundabouts are circular intersections with specific design and traffic control features. These features include yield control of all entering traffic, channelized approaches that deflect traffic flow and appropriate geometric curvature to induce desirable vehicular speeds. Figure 9.7-A illustrates the key components of a roundabout.



ROUNDABOUT ELEMENTS
Figure 9.7-A

9.7.2 Intersection Type Selection

In determining whether to use a roundabout or a more traditional intersection at a site, the designer should consider the following:

1. Safety. The frequency of crashes at an intersection is related to the number of conflict points at an intersection, as well as the magnitude of conflicting flows at each conflict point. A conflict point is a location where the paths of two vehicles, or a vehicle and a bicycle or pedestrian diverge, merge or cross each other. For example, the number of vehicle-vehicle conflict points for four-leg intersections drops from 32 to 8 with roundabouts. Fewer conflict points mean fewer opportunities for collisions.

   The severity of a collision is determined largely by the speed of impact and the angle of impact. The higher the speed and angle of impact, the more severe the collision. Roundabouts reduce in severity or eliminate many severe conflicts that are present in traditional intersections. Consider a roundabout where there is a high number of angle crashes.

2. Construction Costs. The costs of installing roundabouts will vary significantly from site to site. A roundabout may cost more or less than a traffic signal, depending on the amount of new pavement area and the extent of other roadway work required. At some existing unsignalized intersections, a traffic signal can be installed without significant modifications to the pavement area or curbs. In these instances, a roundabout may be costlier to install than a traffic signal, as the roundabout can rarely be constructed without significant pavement and curb modifications.

   However, at new sites, and at signalized intersections that require widening at one or more approaches to provide additional turn lanes, a roundabout can be a comparable or less expensive alternative. While roundabouts typically require more pavement area at the intersection, they may require less pavement width on the upstream approaches and downstream exits if multiple turn lanes associated with a signalized intersection can be avoided. The cost savings of reduced approach roadway widths is particularly advantageous at interchange ramp terminals and other intersections adjacent to grade separations where wider roads may result in larger bridge structures.

3. Movements. Roundabouts tend to treat all movements at an intersection equally. Each approach is required to yield to circulating traffic, regardless of whether the approach is a local street or major arterial. This may result in delay to the major movements that were previously uncontrolled. The delays depend on the volume of turning movements and should be analyzed individually for each approach.

4. Vehicle Delay and Queue Storage. When operating within their capacity, roundabout intersections typically operate with lower vehicle delays than other intersection forms and control types. With a roundabout, it is unnecessary for traffic to come to a complete stop when no conflicts present themselves, deceleration will avoid a conflict. Where there are queues on one or more approaches, traffic within the queues usually continues to move. This is typically more tolerable to drivers than a stopped or standing queue. The performance of roundabouts during off-peak periods is particularly good in contrast to other intersection forms, typically with very low average delays.

5. Signal Progression and Access. It is common practice to coordinate traffic signals on arterial roads to minimize stops and delay to through traffic on the major road. By requiring coordinated platoons to yield to traffic in the circulatory roadway, the introduction of a roundabout into a coordinated signal system may disperse and rearrange platoons of traffic if other conflicting flows are significant, thereby reducing progressive movement. To minimize overall system delay, it may be beneficial to divide the signal system into subsystems separated by the roundabout, assigning each subsystem its own cycle. The traffic performance of the combination roundabout-signal system should be tested in advance with signal systems and roundabout analysis tools. In some cases, total delay, stops and queues will be reduced by the roundabout. The number of available gaps for midblock unsignalized intersections and driveways may also be reduced by the introduction of roundabouts, although this may be offset by the reduced speeds near roundabouts. In addition, roundabouts can enable safe and quick U-turns that can substitute for more difficult midblock left turns, especially where there is no left-turn lane.

6. Environmental Factors. Roundabouts may provide environmental benefits if they reduce vehicle delay and the number and duration of stops compared with an alternative. Even where there are heavy volumes, vehicles continue to advance slowly in moving queues rather than coming to a complete stop. This may reduce noise and air quality impacts and fuel consumption significantly by reducing the number of acceleration/deceleration cycles and the time spent idling. In general, if stop or yield control is insufficient, traffic through roundabouts generates less pollution and consumes less fuel than traffic at fixed-time signalized intersections. However, vehicle-actuated signals typically cause less delay, less fuel consumption and emissions than roundabouts as long as traffic volumes are low.

7. Space Requirements. Roundabouts usually require more space for the circular roadway and central island than the rectangular space available inside traditional intersections. Therefore, roundabouts may have a significant right-of-way impact on the corner properties at the intersection, especially when compared with unsignalized intersections. The dimensions of a traditional intersection are typically comparable to the envelope formed by the approaching roadways. However, to the extent that a comparable roundabout would outperform a signal in terms of reduced delay and thus shorter queues, it will require less queue storage space on the approach legs. If a signalized intersection requires long or multiple turn lanes to provide sufficient capacity or storage, a roundabout with similar capacity may require less space on the approaches. As a result, roundabouts may reduce the need for additional right of way on the links between intersections, at the expense of additional right-of-way requirements at the intersections themselves. The right-of-way savings between intersections may make it feasible to accommodate parking, wider sidewalks, planter strips, wider outside lanes and/or bicycle lanes in order to better accommodate pedestrians and/or bicyclists. Another space-saving strategy is the use of flared approach lanes to provide additional capacity at the intersection while maintaining the benefit of reduced spatial requirements upstream and downstream of an intersection.

   At interchange ramp terminals, paired roundabouts have been used to reduce the number of lanes on crossroad over- and underpasses. In compact urban areas, there are typically signalized intersections at both ends of overpass bridges, necessitating two additional lanes to provide capacity and storage at the signalized intersections.

8. Older Drivers. Roundabouts assist older drivers by reducing the speed at the intersection (i.e., conditions change more slowly allowing for more time to make proper responses), providing less complicated situations and decision-making, judging gaps is easier and mistakes are rarely fatal, providing less demand to accurately judge speeds of traffic, and reducing the required visual scans.

9. Operations and Maintenance Costs. Compared to signalized intersections, a roundabout does not have equipment that requires constant power and maintenance, and regular signal timing updates. Roundabouts, however, can have higher landscape maintenance costs, depending on the degree of landscaping provided on the central island, splitter islands and perimeter. Illumination costs for roundabouts should be considered. The service life of a roundabout is significantly longer, approximately 25 years, compared with 10 years for a typical signal.

10. Traffic Calming. Series of roundabouts can have secondary, traffic calming effects on streets by reducing vehicular speeds. Speed reduction at roundabouts is caused by geometry rather than by traffic control devices or traffic volume. Consequently, speed reduction can be realized at all times of day and on streets of any traffic volume. It is difficult to speed through an appropriately designed roundabout with raised channelization that forces vehicles to physically change direction. In this way, roundabouts can complement other traffic calming measures.

    Roundabouts have also been used successfully at the interface between rural and urban areas where speed limits change. In these applications, the traffic calming effects of roundabouts force drivers to slow and reinforce the notion of a significant change in the driving environment.

11. Aesthetics. Roundabouts offer the opportunity to provide attractive entries or centerpieces to communities. However, hard objects in the central island directly facing the entries are a safety hazard. The portions of the central island and, to a lesser degree, the splitter islands that are not subject to sight-distance requirements offer opportunities for aesthetic landscaping. Pavement textures can be varied on the aprons as well. They can also be used in tourist or shopping areas to facilitate safe U-turns and to demarcate commercial uses from residential areas.

12. Mini-roundabouts. Mini-roundabouts are distinguished from traditional roundabouts primarily by their smaller size and more compact geometry. They are typically designed for negotiating speeds of 15 miles per hour. Inscribed circle diameters generally vary from 45 feet to 80 feet. For most applications, peak-period capacity is seldom an issue, and most mini-roundabouts operate on residential or collector streets at demand levels well below their capacity. It is important, however, to be able to assess the capacity of any proposed intersection design to ensure that the intersection would function properly if constructed.

9.7.3 Locations

The designer should consider providing a roundabout at intersections where one or more of the following apply:

- intersections with high crash rates/high severity rates;
- intersections with complex geometry (e.g., more than four approaches);
- rural intersections with high-speed approaches;
- freeway interchange ramp terminals;
- closely spaced intersections;
- replacement of all-way stops;
- replacement of signalized intersections;
- intersections with high left-turn volumes;
- replacement of two-way stops with high side-street delay;
- intersections with high U-turn movements;
- transitions from higher-speed to lower-speed areas;
- where aesthetics are important; and
- where accommodating older drivers is an objective.

Intersections that may not be good candidates include those with topographic or site constraints that limit the ability to provide appropriate geometry.

9.7.4 Design Considerations

Each roundabout should be designed to meet field conditions. Figure 9.7-B provides general design guidance for roundabouts. For the latest design guidance on roundabouts, review the information provided in NCHRP Report 672, *Roundabouts: An Informational Guide – Second Edition* and guidance found on the FHWA's roundabout website. The traffic designer will determine if a single or multilane roundabout is required.

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Desirable maximum entry design speed	15 to 20 mph	20 to 25 mph	25 to 30 mph
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	45 to 90 ft	90 to 180 ft	150 to 300 ft
Central island treatment	Fully traversable	Raised (may have traversable apron)	Raised (may have traversable apron)

GENERAL ROUNDABOUT CRITERIA
Figure 9.7-B

The following are several design issues that should be addressed in the design of the roundabout:

1. Volumes. In general, a roundabout traffic analysis will not be required if the total entering volume for a four-leg roundabout is less than 10,000 vehicles per day for single

lane roundabouts and 20,000 vehicles per day for two-lane roundabouts. For three-leg roundabouts, use 75% of above volumes. Volumes above these amounts do not automatically warrant increasing the size of the roundabout.

A traffic analysis may be required for any proposed roundabout; particularly if:

- number of entry lanes is not the same for all legs;
- volumes on the legs aren't balanced;
- there is a high percentage of left-turn movements (over 30 percent);
- there is a high volume of pedestrians; or
- there are other geometric considerations that warrant additional analysis (e.g., nearby driveways or intersections).

If a traffic analysis is required, assume the roundabout and each approach leg of the roundabout to operate at no more than 85 percent of capacity (0.85 maximum degree of saturation). Figure 9.7-C provides volume guidance for single and multilane roundabouts.

Volume Range (Sum of Entering and Conflicting Volumes)	**Number of Lanes Required**
0 to 1000 veh/h	• Single-lane entry likely to be sufficient
1000 to 1300 veh/h	• Two-lane entry may be needed • Single-lane may be sufficient based upon more detailed analysis
1300 to 1800 veh/h	• Two-lane entry likely to be sufficient
Above 1800 veh/h	• More than two entering lanes may be required • A more detailed traffic analysis should be conducted to verify lane numbers and arrangements

VOLUME GUIDELINES FOR DETERMINING ROUNDABOUT LANES
Figure 9.7-C

2. Inscribed Circle Diameter. The inscribed circle diameter is the distance across the circle inscribed by the outer curb (or edge) of the circulatory roadway. It is the sum of the central island diameter and twice the circulatory roadway width. The inscribed circle diameter is determined by a number of design objectives, including accommodation of the design vehicle and providing speed control.

   At single-lane roundabouts, the size of the inscribed circle is largely dependent upon the turning requirements of the design vehicle. The inscribed circle diameter typically needs to be at least 105 feet to accommodate a WB-62 design vehicle. Smaller roundabouts can be used for some local street or collector street intersections, where the design vehicle is an S-BUS-40. For locations that must accommodate a WB-67 design vehicle, a larger inscribed circle diameter will be required, typically in the range of 130 feet to 150 feet. In situations with more than four legs, larger inscribed circle diameters may be appropriate. Truck aprons are typically needed to keep the inscribed circle diameter reasonable while accommodating the larger design vehicles.

At multilane roundabouts, the size of the roundabout is usually determined by balancing the need to achieve deflection with providing adequate alignment of the natural vehicle paths. Generally, the inscribed circle diameter of a multilane roundabout ranges from 150 feet to 250 feet. For two-lane roundabouts, a common starting point is 160 feet to 180 feet. Roundabouts with three- and four-lane entries may require larger diameters of 180 feet to 330 feet to achieve adequate speed control and alignment. Truck aprons are typically needed to keep the inscribed circle diameter reasonable while accommodating larger design vehicles.

3. Speeds. The operating speed of a roundabout is widely recognized as one of its most important attributes in terms of safety performance; therefore, the designer should give careful attention to the design speed of a roundabout. Maximum entering design speeds of 20 to 25 miles per hour are recommended for single-lane roundabouts and 25 to 30 miles per hour for multilane roundabouts. These speeds are influenced by a variety of factors, including the geometry of the roundabout and the operating speeds of the approaching roadways.

4. Lane Balance and Continuity. As discussed in Item 1 “Volumes,” the designer should conduct an operational analysis to determine the required number of entry lanes serving each approach to the roundabout. For multilane roundabouts, ensure the design provides the appropriate number of lanes within the circulatory roadway and on each exit.

   The allowed movements assigned to each entering lane affect the overall design. Basic pavement marking layouts are integral to the preliminary design process to ensure that lane continuity is provided. In some cases, the geometry within the roundabout may be dictated by the number of lanes required. Lane assignments should be clearly identified on all preliminary designs in an effort to retain the lane configuration information through the various design iterations.

   In some cases, a roundabout designed to accommodate design year traffic volumes can result in substantially more entering, exiting and circulating lanes than needed in the earlier years of operation. To maximize the potential safety during those early years of operation, the designer may wish to consider a phased design solution that initially uses fewer entering and circulating lanes.

5. Lane Widths. The required width of the circulatory roadway is determined from the number of entering lanes and the turning requirements of the design vehicle. The circulating width should be at least as wide as the maximum entry width and up to 120 percent of the maximum entry width. Typical circulatory roadway widths range from 16 feet to 20 feet for single-lane roundabouts. The designer should avoid making the circulatory roadway width too wide within a single-lane roundabout because drivers may think that two vehicles are allowed to circulate side-by-side.

   At single-lane roundabouts, the circulatory roadway width should be comfortable for passenger car vehicles and should be wide enough to accommodate a design vehicle up to an S-BUS-40 at a small roundabout. A truck apron will often need to be provided within the central island to accommodate larger design vehicles (including the WB-62), but maintain a relatively narrow circulatory roadway to adequately constrain vehicle speeds. Usually, the left-turn movement is the critical path for determining circulatory roadway width. A minimum clearance of 1 foot and preferably 2 feet should be provided between the outside edge of the vehicle's tire track and the curb line.

If the entering traffic for multilane roundabouts is predominantly passenger cars and buses (P and S-BUS-40) and where semi-tractor traffic is infrequent (less than/equal to 10 percent), it may be appropriate to design the width for two P vehicles or a P and S-BUS-40 side by side. If the semi-tractor trailer traffic is frequent, it may be necessary to provide sufficient width for the simultaneous passage of a WB-62 in combination with a P or S-BUS-40 vehicle.

Multilane circulatory roadway lane widths typically range from 14 feet to 16 feet. Use of these values results in a total circulating width of 28 feet to 32 feet for a two-lane circulatory roadway and 42 feet to 48 feet total width for a three-lane circulatory roadway.

6. Alignment of Approaches. The alignment of the approach legs plays an important role in the design of a roundabout. The alignment affects the amount of deflection (speed control) that is achieved, ability to accommodate the design vehicle and visibility angles to adjacent legs. The optimal alignment is generally governed by the size and position of the roundabout relative to its approaches. Figure 9.7-D provides the advantages and tradeoffs for each alignment approach.

The alignment does not have to pass through the center of the roundabout; however, it has a primary effect on the entry/exit design. The optional alignment allows for an entry design that provides adequate deflection and speed control while also providing appropriate view angles to drivers and balancing property impacts/costs.

A roundabout may be designed so that the centerline of each leg passes through the center of the inscribed circle. This location typically allows the geometry of a single-lane roundabout to be adequately designed such that vehicles will maintain slow speeds through both the entries and the exits. The radial alignment also makes the central island more conspicuous to approaching drivers and minimizes roadway modification required upstream of the intersection.

A roundabout may also be designed to offset the centerline of the approach to the left. This alignment will typically increase the deflection achieved at the entry to improve speed control. However, the designer should recognize the inherent tradeoff of a larger radius exit that may provide less speed control for the downstream pedestrian crossing. Especially in urban environments, it is important to have drivers maintain sufficiently low vehicular speeds at the pedestrian crossing to reduce the risk for pedestrians.

	Advantages	Trade-Offs
Alternative 1: Offset Alignment to the Left of Center	• Allows for increased deflection • Beneficial for accommodating large trucks with small inscribed circle diameter — allows for larger entry radius while maintaining deflection and speed control • May reduce impacts to right side of roadway	• Increased exit radius or tangential exit reduces control of exit speeds and acceleration through crosswalk area • May create greater impacts to the left side of the roadway
Alternative 2: Alignment Through Center of Roundabout	• Reduces amount of alignment changes along the approach roadway to keep impacts more localized to intersection • Allows for some exit curvature to encourage drivers to maintain slower speeds through the exit	• Increased exit radius reduces control of exit speeds/ acceleration through crosswalk area • May require a slightly larger inscribed circle diameter (compared to offset-left design) to provide the same level of speed control
Alternative 3: Alignment to Right of Center	• Can be used for large inscribed circle diameter roundabouts where speed control objectives can still be met • Although not commonly used, this strategy may be appropriate in some instances (provided that speed objectives are met) to minimize impacts, improve view angles, etc.	• Often more difficult to achieve speed control objectives, particularly at small diameter roundabouts • Increases the amount of exit curvature that must be negotiated

ENTRY ALIGNMENT
Figure 9.7-D

Approach alignments that are offset to the right of the roundabout's center point typically do not achieve satisfactory results, primarily due to a lack of deflection and lack of speed control that result from this alignment. An offset-right alignment brings the approach in at a more tangential angle and reduces the opportunity to provide sufficient entry curvature. Vehicles will usually be able to enter the roundabout too fast, resulting in more loss-of-control crashes and higher crash rates between entering and circulating vehicles. However, an offset-right alignment alone should not be considered a fatal flaw in a design if speed requirements and other design considerations can be met.

7. <u>Traffic Control</u>. Vehicles entering the roundabout must yield to the traffic within the circle. A YIELD sign is required at the entry along with the appropriate pavement markings. Proper regulatory control, advance warning and directional guidance are required to avoid driver expectancy related problems. Signs should be located where they have the maximum visibility for road users, but a minimal likelihood of even momentarily obscuring pedestrians and bicyclists. Review the roundabout signing and pavement makings criteria in NCHRP Report 672, *Roundabouts: An Informational Guide – Second Edition* and the *MUTCD.* Contact traffic designer for guidance.

8. <u>Splitter Islands</u>. Splitter Islands are designed to separate entering and exiting traffic. A properly designed splitter island also deflects traffic and positions vehicles into a correct alignment to enter the circulatory roadway. This deflection is critical to slowing vehicles before they enter the circulatory roadway. Splitter islands also can be used for pedestrian refuges.

9. <u>Central Island</u>. The central island of a roundabout is the raised, mainly non-traversable area surrounded by the circulatory roadway. It may also include a traversable truck apron. The island is typically landscaped for aesthetic reasons and to enhance driver recognition of the roundabout upon approach. A circular central island is preferred because the constant-radius circulatory roadway helps promote constant speeds around the central island.

10. <u>Curb</u>. When installing a roundabout, including those on an open rural highway, provide concrete curbs at the outer edge of the roundabout and on the approaches.

11. <u>Pedestrians</u>. In urban areas, the designer should anticipate the needs of pedestrians. Pedestrians should not be allowed to enter the central island, but should be directed around the outside of the roundabouts. Locate the crossings on the approaches to the roundabout and set the crossings back from the yield line a minimum of 20 feet. Whenever a raised splitter island is provided, there should also be an at-grade pedestrian refuge. In this case, the crosswalk facilitates two separate moves — curb-to-island and island-to-curb. The exit crossing will typically require more vigilance from the pedestrian and motorist than the entry crossing. It is recommended that all crosswalks be marked.

    Provide special attention to assist pedestrian users who are visually impaired or blind. For example, these users typically attempt to maintain their approach alignment to continue across a street in the crosswalk, because the crosswalk is often a direct extension of the sidewalk. A roundabout requires deviation from that alignment, and attention needs to be given to providing appropriate informational cues to pedestrians regarding the location of the sidewalk and the crosswalk. For example, appropriate landscaping is one method of providing some information. Another is to align the crosswalk ramps perpendicular to the pedestrian's line of travel through the pedestrian refuge.

Special consideration should also be given concerning pedestrian access at multilane roundabouts. Coordinate with the traffic designer on the use of pedestrian signals, crossing restrictions and redirection to adjacent facilities or crossings.

12. Entry Design. The entry is bounded by a curb or edge of pavement consisting of one or more curves leading into the circulatory roadway. It should not be confused with the entry path curve, defined by the fastest vehicular travel path through the entry geometry. At single-lane roundabouts, a single entry curb radius is typically adequate; for approaches on higher speed roadways, the use of compound curves may improve guidance by lengthening the entry arc.

    The entry curb radius is an important factor in determining the operation of a roundabout because it affects both capacity and safety. The entry curb radius, in conjunction with the entry width, the circulatory roadway width and the central island geometry, controls the amount of deflection imposed on a vehicle's entry path. Excessively large entry curb radii have a higher potential to produce faster entry speeds than desired.

13. Exit Design. The exit curb radii are usually larger than the entry curb radii in order to minimize the likelihood of congestion and crashes at the exits. This, however, is balanced by the need to maintain slow speeds through the pedestrian crossing on exit. The exit design is also influenced by the design environment (urban versus rural), pedestrian demand, the design vehicle and physical constraints.

14. Bicyclists. Research of roundabouts in the United States has not found any substantial safety problems for bicyclists, as indicated by few crashes being reported in detailed crash reports. Nevertheless, roundabouts slow drivers to speeds more compatible with bicycle speeds, while reducing high-speed conflicts and simplifying turn movements for bicyclists. Typical on-road bicyclists speeds are 12 to 20 miles per hour, so designing roundabouts for circulating traffic to flow at similar speeds will minimize the relative speeds between bicyclists and motorists and thereby improve safety and usability for cyclists. Bicyclists require particular attention in two-lane roundabout design, especially in areas with moderate to heavy bicycle traffic. On multi-lane roundabouts, a bicycle path that is separate and distinct from the circulatory roadway is preferred (e.g., a shared bicycle-pedestrian path of sufficient width and appropriately marked to accommodate both types of users around the perimeter of the roundabout).

15. Trucks. Roundabouts should always be designed for the largest vehicle that can be reasonably anticipated (the design vehicle). For single-lane roundabouts, this may require the use of a mountable truck apron around the perimeter of the central island to provide the additional width needed for the off-tracking of the trailer wheels. At multi-lane roundabouts, large vehicles may track across the whole width of the circulatory roadway to negotiate the roundabout.

16. Illumination. Lighting of roundabouts serves two main purposes:

    - it provides visibility from a distance for users approaching the roundabout; and
    - it provides visibility of the key conflict areas to improve users' perception of the layout and visibility of other users within the roundabout.

Illumination is required for all roundabouts, including those in rural environments. Where lighting is not provided, additional delineation for the entry islands and splitter islands should be provided so that they can be correctly perceived by day and night.

In areas where only the roundabout is illuminated (no lighting is provided on the approach roadways), the scope of illumination needs to be carefully considered. Any raised channelization or curbing should be illuminated. A gradual illumination transition zone should be provided beyond the final trajectory changes at each exit. Review the illumination criteria in NCHRP Report 672, *Roundabouts: An Informational Guide – Second Edition* and contact the traffic designer for guidance.

17. Transit. Transit considerations at a roundabout are similar to those at a conventional intersection. If the roundabout has been designed using the appropriate design vehicle, a bus should have no physical difficulty negotiating the intersection. To minimize passenger discomfort, if the roundabout is on a bus route, it is preferable that scheduled buses are not required to use a truck apron if present. Locate bus stops to minimize the probability of vehicle queues spilling back into the circulatory roadway. This typically means that bus stops located on the far side of the intersection need to have pullouts or be further downstream than the splitter island.

18. Rail Crossings. Rail crossings through or near a roundabout may involve many of the same design challenges as at other intersections and should be avoided if better alternatives exist. In retrofit, the rail track may be designed to pass through the central island, or across one of the legs. Queues spilling back from a rail blockage into the roundabout can fill the circulatory roadway and temporarily prevent movement on any approach.

9.8 MEDIAN OPENINGS

9.8.1 Location

Median openings are provided on all divided highways with limited control of access or uncontrolled access provided the openings are sufficiently spaced. The designer should evaluate the following recommended minimum spacing when determining the location for a median opening:

1. Facilities with Controlled Access. U-turn median openings may be provided on freeways having non-barrier medians and where needed for the proper operation of police and emergency vehicles. They may also be provided for equipment engaged in routine maintenance and rest area maintenance where rest areas are across the median from each other.

2. Facilities with Partial Access Control. On rural divided highways, median openings are provided at most public highways and streets (site specific). Median openings are generally provided for major traffic generators (e.g., industrial or commercial parks). Median openings for driveway access should not be spaced closer than 1500 feet to 2000 feet apart.

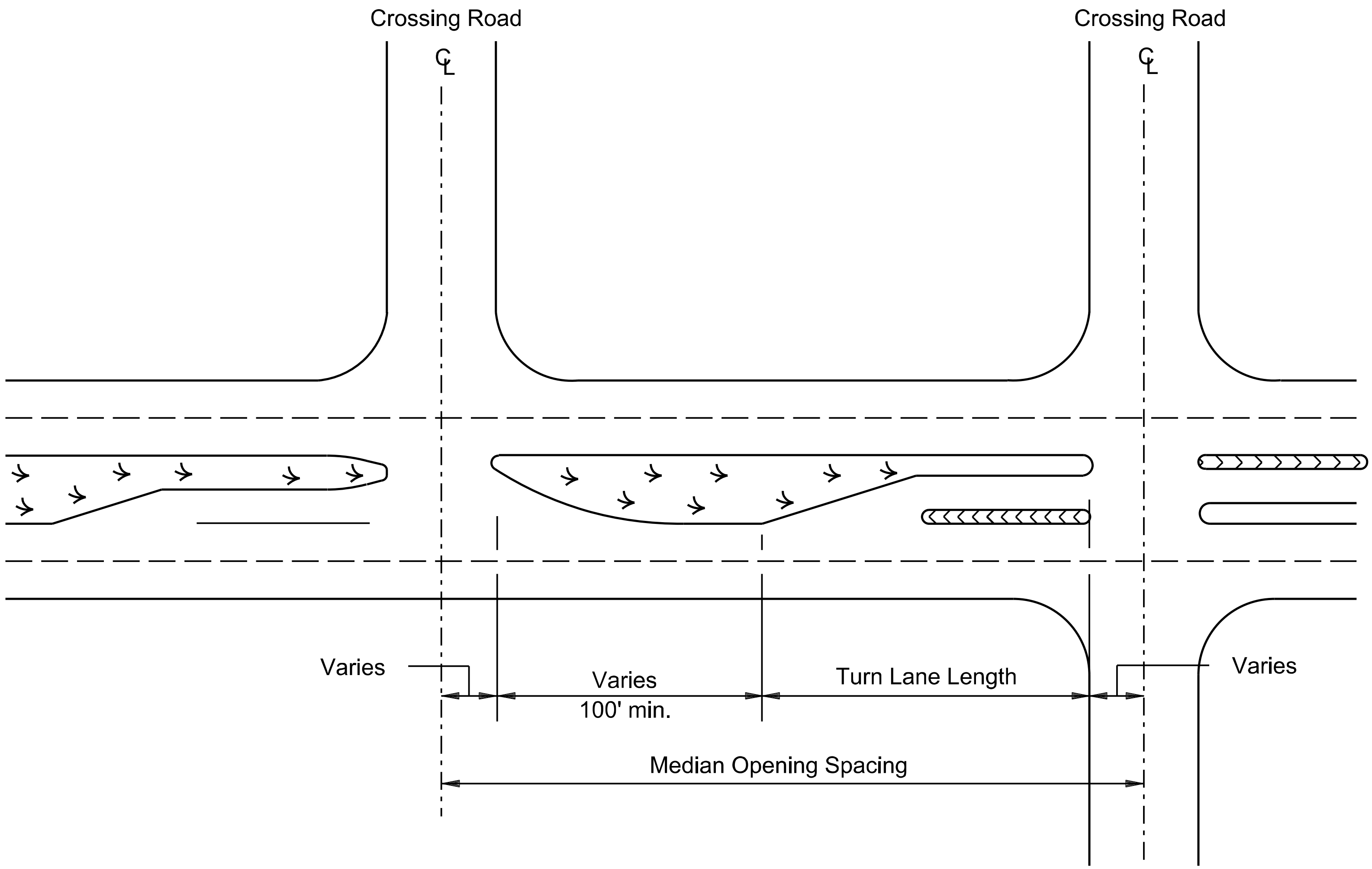
3. Facilities with Uncontrolled Access. Provide a median opening at all public highways and streets. Median openings for driveway access should not be less than 1000 feet to 1500 feet apart.

In addition to the above criteria, the location of median openings should be consistent with the following design considerations:

1. Signal Coordination. Median openings (both signalized and unsignalized) must not impair the traffic signal coordination of the overall facility.

2. Sight Distance. Do not locate median openings in areas of restricted sight distance (e.g., on a horizontal curve or near the apex of a crest vertical curve). Section 4.4 discusses the minimum intersection sight distances that should be available at a median opening.

3. Turn Lane Length. Provide median openings only if the full length of a left-turn lane can be provided and if the beginning of the turn lane taper is at least 100 feet from the median nose of the previous intersection. See the schematic in Figure 9.8-A. Determine the length of the left-turn lane using the criteria in Section 9.5.2.2.



LOCATIONS OF MEDIAN OPENINGS ON NON-FREEWAYS
Figure 9.8-A

9.8.2 Design

Figure 9.8-B presents a general figure for the design of a median opening. The following will apply to the design of median openings:

1. Design Vehicle. The designer should select the largest vehicle that will be making the turn with some frequency as the design vehicle for median openings. The process for the selection of the design vehicle is the same as for a right-turning vehicle; see Section 9.2.5.

2. Encroachment. The desirable design will allow the design vehicle to make a left turn and to remain entirely within the through inside lane of the divided facility. In addition, the turning vehicle should be no closer than 2 feet to the inside curb or inside edge of traveled way. However, depending on traffic control or available intersection sight distance, it would be acceptable for the design vehicle to occupy both travel lanes (e.g., a vehicle stopped at a signalized intersection exiting from a side street may turn left on any green signal while encroaching on both travel lanes of the facility being turned onto); see Figure 9.8-B.

3. Length of Opening. The length of a median opening should properly accommodate the turning path of the design vehicle. The minimum length is the largest of the following:

   - approach width plus the width of shoulders, including crossroad median width;
   - the length based on the selected design vehicle; or
   - 40 feet.

   Evaluate each median opening individually to determine the proper length of opening. Consider the following factors in the evaluation:

   a. Turning Templates. Check the proposed design with the turning template for the selected design vehicle. Give consideration to the frequency of the turn and to the encroachment onto adjacent travel lanes or shoulders by the turning vehicle.

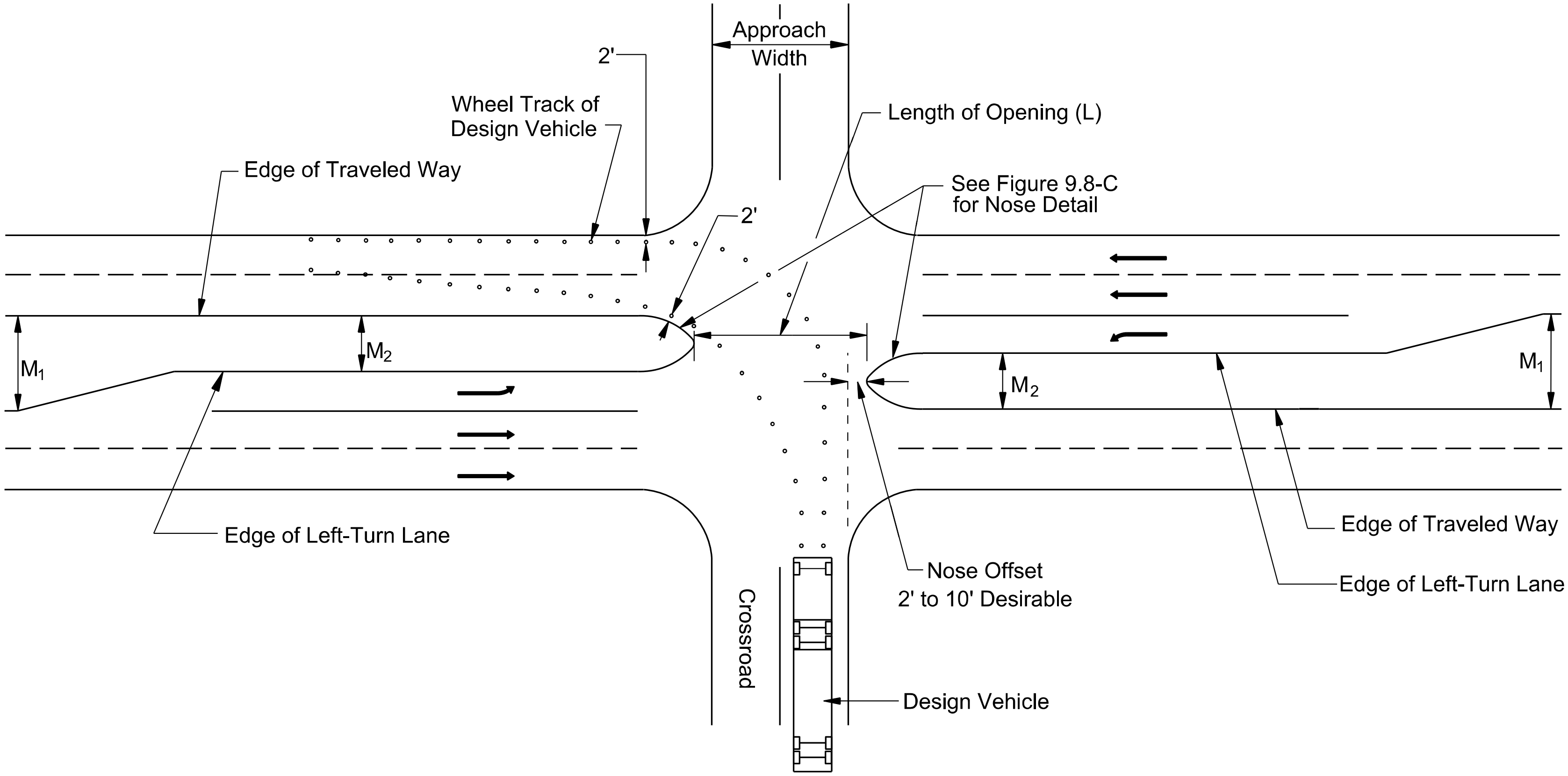
   b. Nose Offset. At four-leg intersections, traffic passing through the median opening (going straight) will pass the nose of the median end (semicircular or bullet nose). To provide a sense of comfort for these drivers, the offset between the crossroad through travel lane (extended) and the median nose should be at least 2 feet.

   c. Lane Alignment. Provide a design where the lanes line up properly across the intersection.

   d. Location of Crosswalks. Desirably, pedestrian crosswalks will intersect the median nose to provide some refuge for pedestrians. Therefore, the median opening design should be coordinated with the location of crosswalks.

   e. Traffic Control. See the *MUTCD* for the design of intersection signing, striping and traffic control.

MEDIAN OPENING DESIGN
Figure 9.8-B



Note: See discussion in Section 9.8.2 for minimum L criteria.

4. Median Nose Design. The shape of the nose at median openings is determined by the width of the median (M_1) or (M_2). The two basic types of median nose designs are the semicircular design and bullet-nose design. The following summarizes their usage:

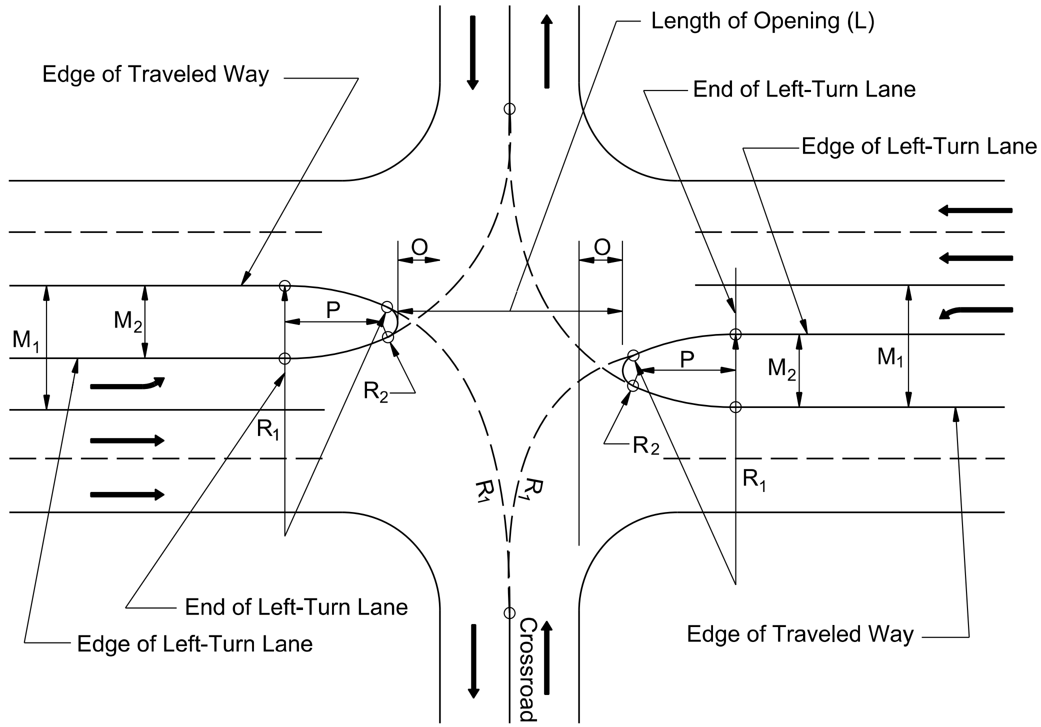
   - For medians up to 4 feet in width, there is little operational difference between the two designs.
   - The semicircular design is generally acceptable for median widths (M_1) up to 10 feet.
   - For medians (M_1) wider than 10 feet, use the bullet-nose design. Also, use this design for the divisional island remaining after locating a left-turn lane in the median.
   - As medians become successively wider, the minimum length of the median opening becomes the governing design control.

   For the bullet-nose design, a compound curvature arrangement should be used. Figure 9.8-C provides the typical details for a median opening with a bullet-nose design.

5. U-turns. Median openings are sometimes used to accommodate U-turns on divided highways. Preferably, a vehicle should be able to begin and end the U-turn on the inner lanes next to the median. Figure 9.8-D provides the minimum median widths for U-turn maneuvers for various design vehicles and various levels of encroachment. Check the U-turn design with the applicable turning template.

6. Sight Distance. Check all median openings for the applicable sight distance criteria; see Section 4.4.

7. Wide Medians. Where the median is wider than 75 feet, treat the design as two separate intersections.



MEDIAN NOSE DESIGN
Figure 9.8-C

L = *Length of median opening. See discussion in Section 9.8.2 for minimum L values.*

M_1 = *Median width measured between the two edges of the inside travel lanes.*

M_2 = *Width of divisional island (flush, raised-curb, depressed) remaining after the width of the left-turn (if present) has been subtracted from the median width (M_1).*

O = *Nose offset.*

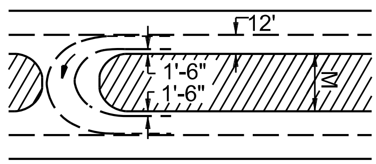
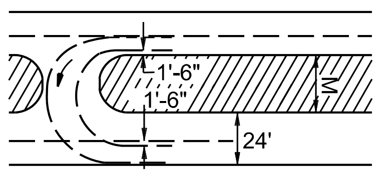
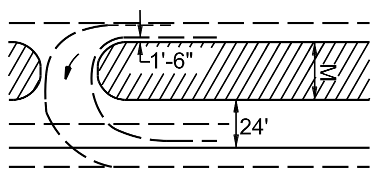
P = *As shown in figure.*

R_1 = *Variable, based on design vehicle and median width (M_2).*

R_2 = *M_2/5 to edge of left-turn lane, where present.*

R_2 = *M_1/5 to edge of traveled way where a left-turn lane is not present.*

R_2 is typically rounded up to the next highest whole number.

Type of Maneuver		**M - Min. width of median for design vehicle (ft)**		
		P	**S-BUS-40**	**WB-62**
		Length of design vehicle (ft)		
		19	**40**	**55**
Inner Lane to Inner Lane		30	63	71
Inner Lane to Outer Lane		18	51	59
Inner Lane to Shoulder		8	41	49

Note: The selected design vehicle will affect the length of the median opening.

MINIMUM WIDTHS NEEDED FOR U-TURNS
Figure 9.8-D

SPACER PAGE

9.9 RAILROAD/HIGHWAY GRADE CROSSINGS

9.9.1 Railroad Crossings Near Intersections

These design guidelines apply to all highway improvement projects where the route is adjacent and parallel to a railroad. Where an at-grade railroad crossing is within 200 feet of an intersection, the design should address efforts to keep vehicles from stopping or storing on the tracks. This applies to either signal- or stop-controlled intersections. The following factors should be identified and considered during the planning stages:

1. Clear Storage Distance. Consider alternative designs that provide a minimum distance of 75 feet between the proposed intersection stop line and a point 6 feet from the closest rail.

2. Space for Vehicular Escape. On the far side of any railroad crossing, consider providing an escape area for vehicles (e.g., shoulder with curb and gutter behind the shoulder, flush medians, flush-corner islands, right-turn acceleration lanes, improved corner radii).

3. Conflicting Commercial Access. Left-turn vehicular movements that may inhibit the clearance of queued traffic on the approaches to railroad tracks should be discouraged. If entrances on the street approach exist, consider using design features that would eliminate the problems (e.g., left-turn lane, raised-curb median).

4. Restricted Intersection Capacity. During periods of frequent railroad preemption of traffic signals, consider the effects of reduced traffic flow, lack of progression on the street paralleling the tracks and traffic backups. Use available computer programs to analyze different capacity and operational scenarios and to recommend any countermeasures.

5. Protected Left-Turn Storage. On the street that parallels the tracks, analyze the storage length needed for left turns into the side street and across the tracks during preemption of the traffic signals. Without the proper storage length available, this could cause backups into the through lanes.

6. Right-Turn Lanes. On the street that runs parallel to the railroad and where an actuated NO RIGHT TURN sign is proposed in conjunction with railroad preemption, the designer should consider providing a right-turn lane for the right-turn movement across the tracks. The auxiliary lane provides a refuge for right-turning vehicles during railroad preemption and eliminates the problem of traffic temporarily blocking the through lanes.

7. Side Street Left-Turn Lane Capacity. On streets that cross railroad tracks, provide sufficient left-turn storage lengths. This will avoid the problem of left turns spilling out onto through lanes and blocking the through lanes.

9.9.2 Interconnection Traffic Signal System Design

9.9.2.1 General

Where a signalized intersection is located within 200 feet of a railroad grade crossing or where traffic frequently queues onto the tracks, the normal sequence of the traffic signals should be preempted upon approach of trains to avoid entrapment of vehicles on the crossings. The primary focus of the design of intersections where a railroad grade crossing is within 200 feet should be to provide adequate storage area for vehicles between the track and intersection and to keep vehicles from stopping on the tracks while waiting for a green signal at the intersection.

9.9.2.2 Coordination

Close coordination between the Department and the railroad company is required to ensure that the railroad warning signal and the traffic signal are performing as a system. The traffic designer will be responsible for reviewing the interconnected traffic signal design.

9.9.3 Sight Distance

If passive traffic controls are used at the railroad crossing, then the designer should address the following sight distance applications:

1. Case A. This sight distance is required by the motorist that will allow the driver to either pass through the grade crossing prior to the train's arrival or to stop the vehicle prior to encroachment in the crossing area.

2. Case B. A motorist who is stopped at the crossing and then decides to cross the tracks requires this sight distance.

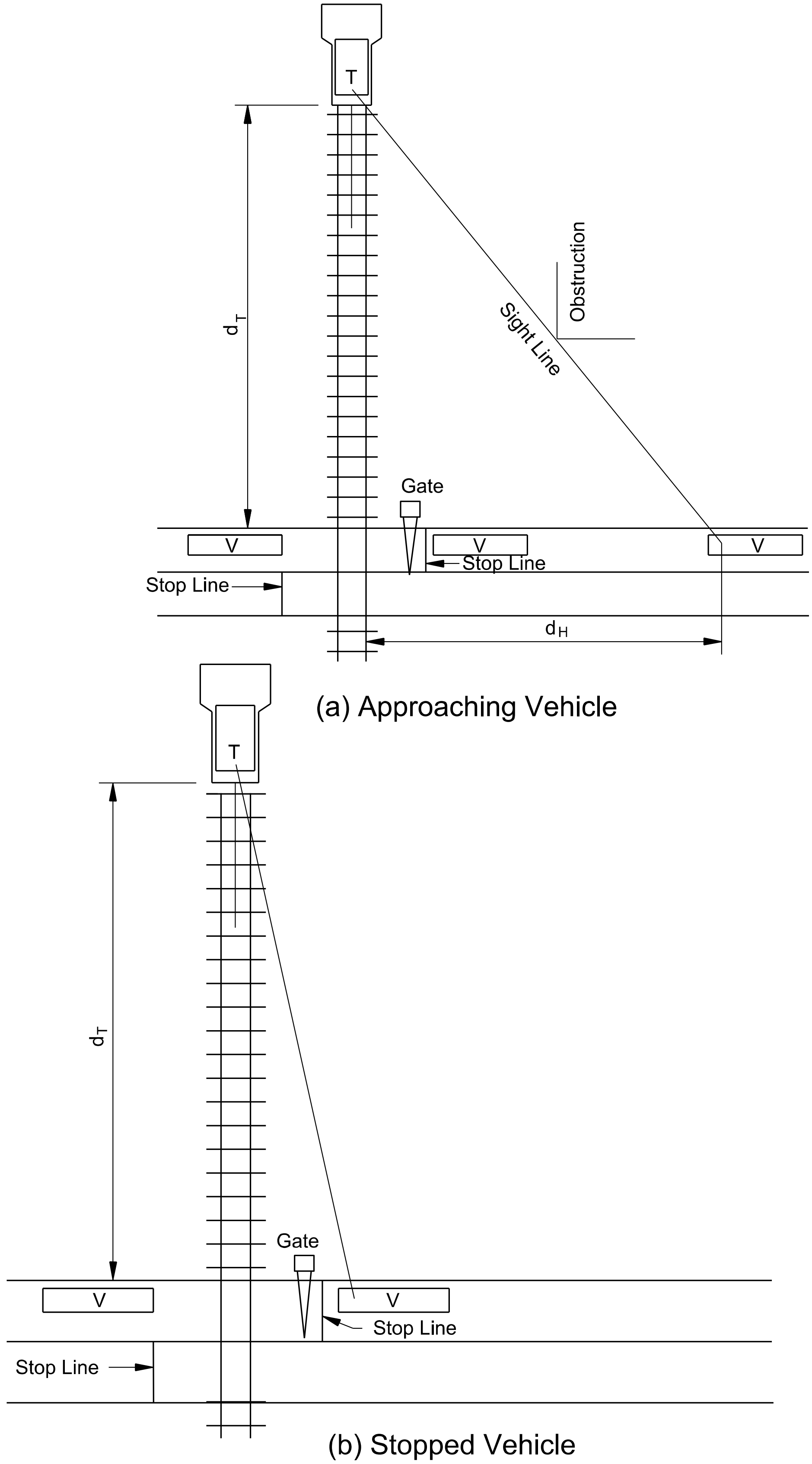
Figure 9.9-A illustrates these two sight distance measurements. Figure 9.9-B provides the sight distance values for both Case A and Case B. These criteria are appropriate for railroad/ highway intersections at 90 degrees; adjust these distances for skewed intersections. Additional guidance on railroad crossing sight distance can be found in AASHTO *A Policy on Geometric Design of Highways and Streets*.

9.9.4 Horizontal Alignment

Where practical, the alignment of the highway and railroad crossing should intersect at 90 degrees, and neither the highway nor the railroad should be on a horizontal curve. If these objectives are met, this will enhance driver safety and comfort, reduce maintenance problems and improve roadway rideability. The designer should review Chapter 5 "Horizontal Alignment" for SCDOT criteria on horizontal alignment for highways.

9.9.5 Vertical Alignment

Desirably, the highway will be relatively level where it crosses the railroad. Where vertical curves are provided, they should meet SCDOT criteria for vertical alignment presented in Chapter 6 "Vertical Alignment." Figure 9.9-C presents the minimum design for vertical alignment at railroad crossings to prevent low-clearance vehicles from bottoming out on the tracks. This design should be provided unless railroad track superelevation dictates otherwise.



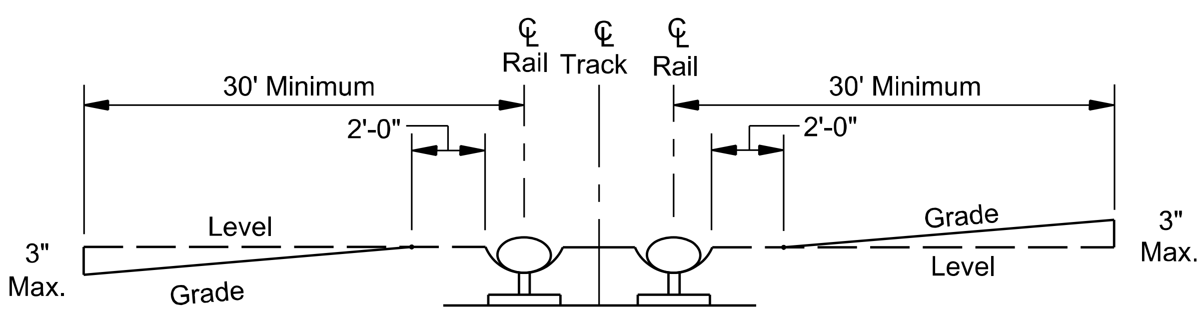
SIGHT LINES AT RAILROAD CROSSING
Figure 9.9-A

Train Speed (mph)	Case B Departure from stop (0 mph)	Case A Moving Vehicle Vehicle Speed (mph)							
		10	**20**	**30**	**40**	**50**	**60**	**70**	**80**
		Distance along railroad from crossing, d_T(ft)							
10	255	155	110	102	102	106	112	119	127
20	509	310	220	203	205	213	225	239	254
30	794	465	331	305	307	319	337	358	381
40	1019	619	441	407	409	426	450	478	508
50	1273	774	551	509	511	532	562	597	635
60	1528	929	661	610	614	639	675	717	763
70	1783	1084	771	712	716	745	787	836	890
80	2037	1239	882	814	818	852	899	956	1017
90	2292	1394	992	915	920	958	1012	1075	1144
	Distance along highway from crossing, d_H(ft)								
All		69	135	220	324	447	589	751	931

SIGHT DISTANCE AT RAILROAD CROSSINGS
Figure 9.9-B

9.9.6 Grade Crossing Surface

The railroad company in coordination with the Railroad Projects Office will select the grade crossing surface type. The designer should contact the Railroad Projects Office to determine what measures will be required to connect to the grade crossing.



PROFILE AT RAILROAD/HIGHWAY GRADE CROSSINGS
Figure 9.9-C

9.10 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. *Highway Capacity Manual 2010*, Transportation Research Board, 2010.

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Chapter 10

INTERCHANGES

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 10
INTERCHANGES

An interchange is a system of ramps in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways on different elevation levels. The operational efficiency, capacity, safety and cost of the highway facility are largely dependent upon its design. This chapter provides guidance in the design of interchanges including type, selection, operations, spacing, freeway/ramp terminals, ramps and ramp/crossroad terminals.

10.1 GENERAL

10.1.1 Interchange Warrants

High cost and environmental impact require interchanges be provided only after careful consideration of their merits. Because of the variance in specific site conditions, SCDOT has not adopted specific interchange warrants. When determining the need for an interchange or grade separation, consider the following:

1. Access Control. The following will apply:

   a. Full Access Control. On all fully access-controlled facilities, intersecting crossroads must be terminated, rerouted, grade separated or an interchange provided. The importance of the continuity of the crossroad, the feasibility of an alternative route, traffic volumes, construction costs, environmental impacts, etc., are evaluated to determine which option is most beneficial. Interchanges generally are provided at:

      - all freeway-to-freeway crossings;
      - all major highways, unless determined inappropriate; and
      - other highways based on the anticipated demand for regional access.

   b. Partial Access Control. On facilities with partial access control (expressways), intersections with public roads will be accommodated by an interchange or with an at-grade intersection. Generally, it will be rare that a grade separation or an interchange will be provided where the facilities are not access controlled, topography limits the potential to provide grade separation or costs are prohibitive. However, an interchange may still be a viable option for high-volume intersecting roads when considering Items 2 through 6.

   c. No Access Control. An interchange will rarely be warranted on a facility with no access control. The need for an interchange will be determined on a case-by-case basis emphasizing cost effectiveness, safety and operations. A road-user benefit analysis will generally be required to determine the economic feasibility of an interchange; see Item 4. However, this analysis alone is not a sufficient justification for the provision of an interchange.

2. Safety. In special cases, consider the crash-reduction benefits of an interchange at an existing intersection that exhibits extremely high-crash frequencies or rates.

3. Site Topography. Where access is necessary, the topography may dictate an interchange or a grade separation rather than an intersection.

4. Road-User Benefits. If an analysis reveals that road-user benefits over the service life of the interchange will exceed costs, then an interchange may be considered. The designer must consider all costs including right of way, construction, maintenance and user costs in the analysis. For additional guidance, see the AASHTO publication *User and Non-User Benefit Analysis for Highways*.

5. Reduction of Bottlenecks. Insufficient capacity at the intersection of heavily traveled routes often results in significant congestion on one or all approaches. The inability to provide essential capacity with an at-grade facility may warrant an interchange where development and available right-of-way permit. Even on facilities with partial control of access, the elimination of random signalization contributes greatly to improvement of free-flow characteristics.

6. Traffic Volumes. Although there are no specific traffic volumes that warrant an interchange, consider providing an interchange where the traffic volumes at an intersection are at or near capacity and where other improvements are not practical. Consider providing an interchange where the level of service (LOS) at an intersection is unacceptable and the intersection cannot be redesigned to operate at an acceptable LOS.

10.1.2 New/Reconstructed Interchanges

In general, all new and/or modified access points should be minimized on existing fully access-controlled facilities. Each entrance and exit point on the mainline, including locked gate access (e.g., utility opening), is defined as an access point. A modified access includes changes in an existing interchange configuration although the number of access points may not change.

The Department must demonstrate that an additional access point or revision is required for regional traffic demand and not just to solve local system needs or problems. The Interstate and other freeway facilities, including the interchange crossroad and ramps, should not be allowed to become a part of the local circulation system, but should be maintained to handle regional traffic demands.

SCDOT and FHWA must approve all proposed changes in interchange configurations on the Interstate System, even if the number of access points does not change. See the *Federal Register*, Vol. 74, No. 165, August 27, 2009. The FHWA *Interstate System Access Information Guide* provides guidance on the procedures for documenting these requests.

10.1.3 Grade Separation versus Interchange

Section 17.4.2 discusses the justification for a grade separation and general design considerations. Section 17.4.2 also presents criteria for determining if the major road should pass over or under the crossroad. Once it has been determined to provide a grade-separated crossing, the need for access between the two roadways must be evaluated to determine if an interchange is appropriate. The following lists several guidelines to consider in the evaluation:

1. Functional Classification. Provide an interchange at all freeway-to-freeway crossings. On fully access-controlled facilities, provide an interchange with all major highways, unless this is determined inappropriate for other reasons (e.g., terrain). Consider providing interchanges to other highways, if practical.

2. Site Conditions. Site conditions that may be adaptable to a grade separation may not always be conducive to an interchange. Restricted right of way, environmental concerns, rugged topography, etc., may restrict the practical use of an interchange.

3. Interchange Spacing. Freeway operations are improved with increased interchange spacing. See Section 10.3.1 for guidance on interchange spacing. If these criteria cannot be met, this may favor the use of a grade separation rather than an interchange.

4. Operations. Grade-separated facilities without ramps will allow traffic to cross the facility. All drivers desiring to turn onto the crossroad must use other locations to make their desired moves. This will often improve the operations of the major facility by concentrating the access to a few strategically placed locations. Concentration of the access movements at specific locations will affect the operation of the interchange.

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10.2 INTERCHANGE TYPES AND SELECTION

10.2.1 General

SCDOT uses five basic interchange types — the diamond, cloverleaf, partial cloverleaf, three-leg and directional. These interchange types, and variations within each type, permit adaptation to traffic needs, available right of way, terrain and cultural features. The following sections discuss these basic interchange types and the design elements for laying out the interchange. Each interchange must be designed to fit the individual site considerations. The final design may be a minor or major modification of one of the basic types or may be a combination of two or more basic types.

Section 10.2.13 discusses the selection of an interchange type.

10.2.2 Conventional Diamond

The conventional diamond is the simplest and most common interchange type. Diamonds include one-way diagonal ramps in each quadrant and two intersections at the crossroad. With proper treatments at the crossroad (e.g., intersection capacity, adequate storage distance between ramps, vertical and horizontal alignment), the diamond is often the best interchange choice where the intersecting road is not access controlled. Figure 10.2-A illustrates a typical diamond interchange without frontage roads. Figure 10.2-B illustrates a typical diamond interchange with frontage roads. Some of the advantages and disadvantages of a conventional diamond include:

Advantages

1. All exits from the mainline occur before reaching the crossroad structure and entrances occur after the structure. This conforms to driver expectancy and therefore minimizes confusion.

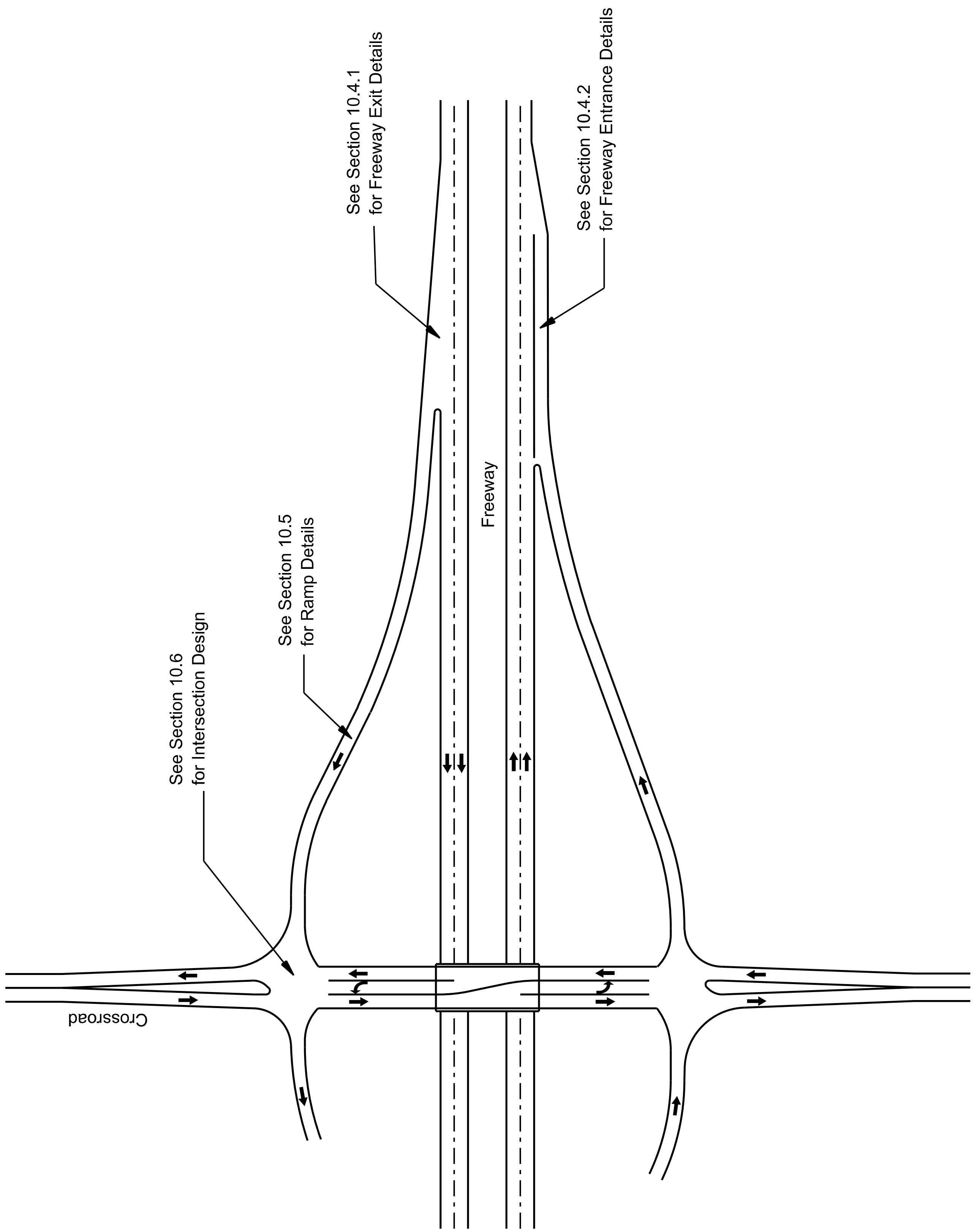
2. All traffic can enter and exit the mainline at relatively high speeds. The operational maneuvers are normally uncomplicated.

3. At the crossroad, adequate sight distance can usually be provided, and the operational maneuvers are consistent with other intersections on the crossroad.

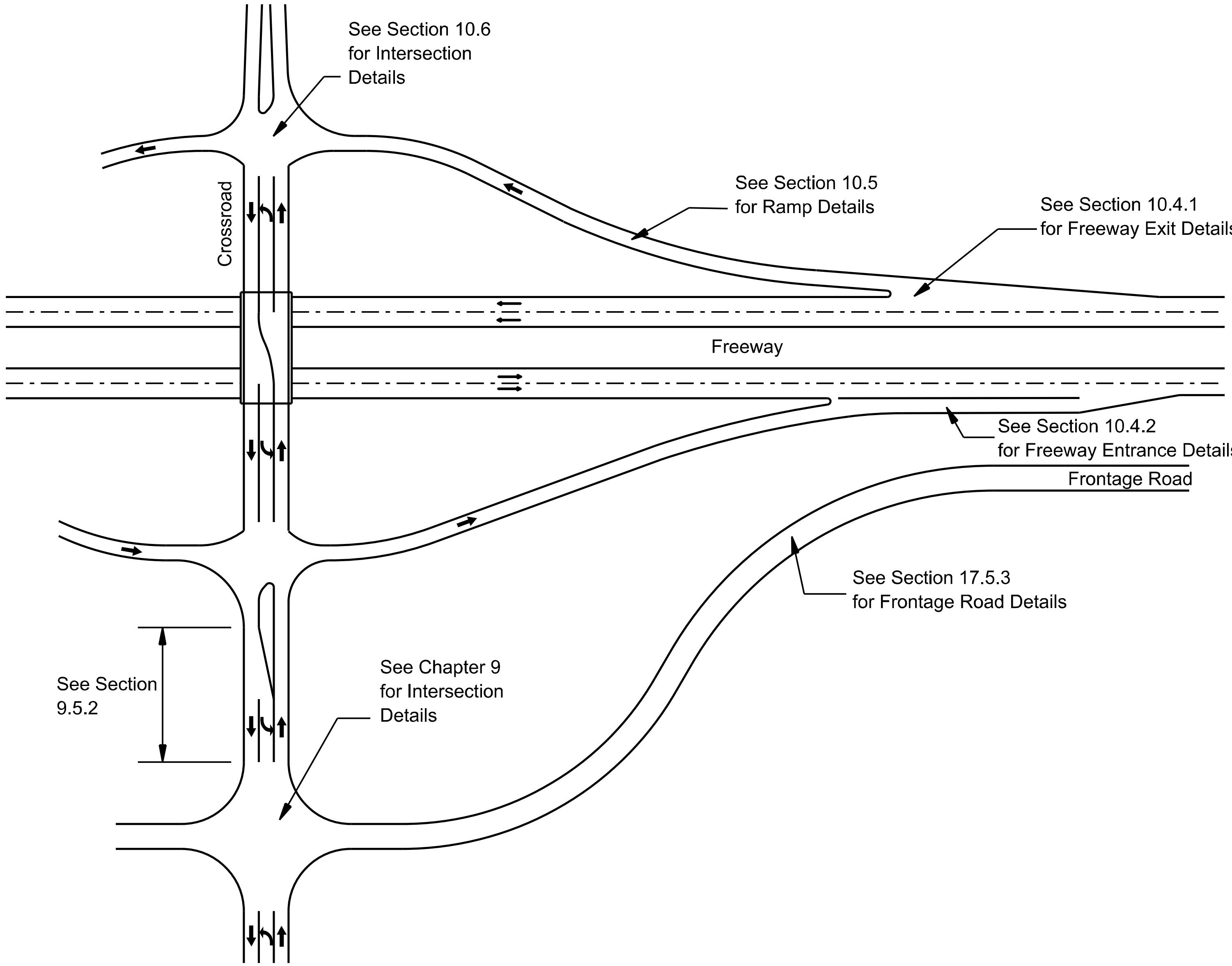
4. The diamond requires less right of way than other interchange types.

5. Their common usage has resulted in a high level of driver familiarity.

6. Typically, it is the least expensive of all interchange types.



DIAMOND INTERCHANGE
(Without Frontage Roads)
Figure 10.2-A



DIAMOND INTERCHANGE
(With Frontage Roads)
Figure 10.2-B

Disadvantages

1. Introduces two at-grade intersections at the crossroad. The designer needs to consider intersection spacing, sight distance, left-turn storage between ramps, signal coordination, etc.

2. Traffic is subject to stop-and-go operations rather than free flow.

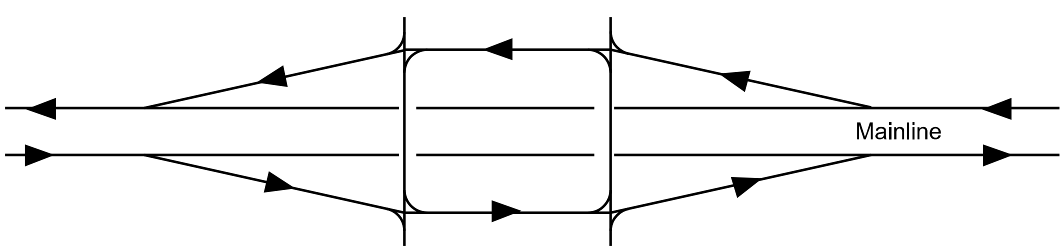
3. In suburban and urban areas, signalization is generally required at the crossroad intersections. These signals may need to be interconnected for progression. Signalization may also produce vehicular platoons entering the freeway, which may cause congestion in the freeway/ramp merge area.

4. A diamond requires right of way in all four quadrants of the interchange.

5. A diamond has a greater potential for wrong-way entry onto the ramps than, for example, a full cloverleaf.

10.2.3 Split Diamond

A variation of the conventional diamond is a split diamond interchange; see Figure 10.2-C. Split diamonds are normally used in urban or suburban areas where the designer desires to provide access to two crossing roadway facilities that are spaced less than one mile apart. Normally, separate interchanges cannot be located within this distance without creating substandard geometric conditions and/or weaving problems without the use of collector-distributor (C-D) roads. It is desirable to make the connecting roadways (between the two crossroads) one-way with control of access. Split diamonds have an undesirable feature in that traffic leaving the freeway cannot return at the same interchange point and continue in the same direction.



SPLIT DIAMOND INTERCHANGE
Figure 10.2-C

10.2.4 Compressed Diamond

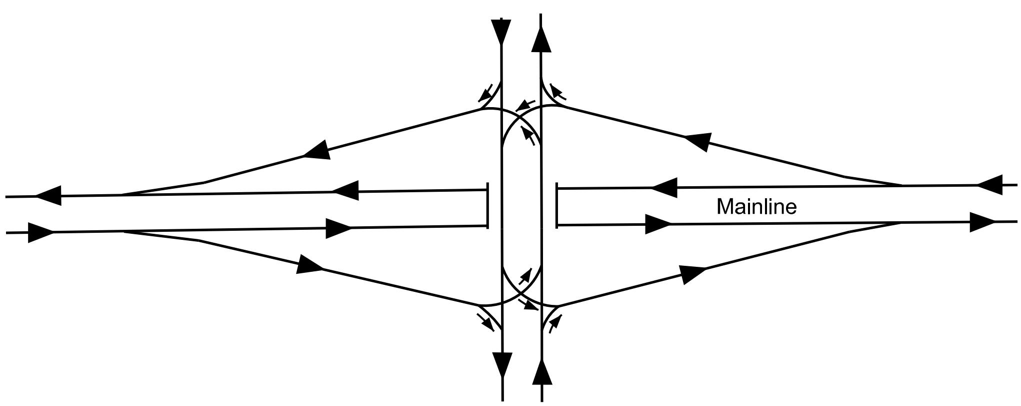
A compressed diamond is similar to the conventional diamond except that the ramp termini on the crossroad are spaced approximately 500 feet to 700 feet apart. Figure 10.2-D presents a schematic of a compressed diamond interchange without frontage roads. This design type is generally only used in urban areas where a diamond interchange is appropriate, but right of way or other environmental features preclude the use of the conventional diamond. Although operationally a compressed diamond is similar to a tight diamond or single-point diamond discussed in Section 10.2.6, they have significant differences. Some of the advantages and disadvantages of the compressed diamond include:

Advantages

1. Generally, less right of way is required than that for a conventional diamond.

2. The open pavement area at the intersection is significantly less than that for a single-point diamond.

3. The grade separation structure is smaller than that for a single-point diamond, retaining walls and/or embankments are less expensive, and construction costs are lower.



COMPRESSED DIAMOND INTERCHANGE
Figure 10.2-D

Disadvantages

1. Left-turn lanes between the ramp termini usually need to be overlapped (i.e., side-by-side opposing left-turn lanes). Consequently, the cross section of the crossroad is generally wider than a conventional diamond.

2. Signal timing and interconnection are necessary to eliminate left-turn queues from overlapping upon each other and causing gridlock.

3. Because of the spacing between intersections and the three-phase signalization, efficient signal coordination is difficult.

4. The length of access control on the crossroad may be more extensive than for a conventional diamond.

5. A diamond has a greater potential for wrong-way entry onto the ramp than a regular diamond.

6. There is the potential for the left turn lane queue to block ramp access.

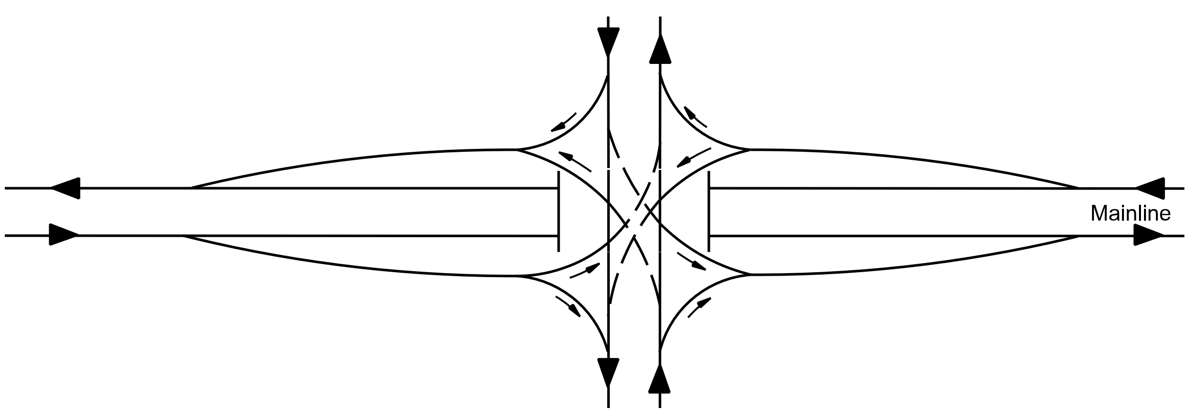
10.2.5 Tight-Urban Diamond

The tight-urban diamond interchange is similar to the compressed diamond except that the spacing between the two intersections is only 250 feet to 350 feet apart. In addition, only one signal controller is required for the tight-urban diamond versus two for the compressed diamond. In addition to the advantages of the compressed diamond, the tight-urban diamond intersections operate as a single intersection for signal control. The signal timing and phasing scheme typically precludes the need for storage of vehicles between the two intersections. All vehicles are stored external to the two intersections on the cross street and on the exit ramp approaches.

The operations and geometrics along the freeway approach for tight-urban diamond and single point diamond interchanges are essentially identical. The advantages of the tight-urban diamond compared to single-point diamond are that structural costs are typically lower for the tight-urban diamond and that pedestrians, bicyclists and frontage roads can be easier accommodated with the tight-urban diamond.

10.2.6 Single-Point Diamond

The single-point diamond interchange (SPDI) is a variation of a tight-urban diamond except instead of two intersections there is only one for the SPDI. This interchange offers improved traffic-carrying capabilities, safer operations and reduced right of way needs under certain conditions when compared with other interchange configurations. The distinguishing feature of this interchange is the convergence of all through and left-turning movements into a single, large signalized intersection area. Figure 10.2-E illustrates a schematic of a SPDI. Some of its advantages and disadvantages include:



SINGLE-POINT DIAMOND INTERCHANGE
Figure 10.2-E

Advantages

1. The SPDI only requires one intersection instead of two intersections at a typical diamond.

2. It allows for better traffic signal progression on the crossroad.

3. The SPDI can increase interchange capacity and alleviate storage problems from two closely spaced intersections on the crossroad.

4. Opposing left turns operate to the left of each other so that their paths do not cross each other.

5. Less right of way is required than any other interchange type.

6. At the intersection of the ramps with the crossroad, the design typically includes flatter curves for turning radii, which allows left turns to be completed at higher speeds.

Disadvantages

1. Special pavement markings and a centrally located diamond-shaped island are required to guide the left-turning drivers through the intersection.

2. There is a significantly wider pavement area for pedestrians to cross, and the SPDI may create greater delays in traffic when compared to the conventional diamond.

3. Because of wide pavement areas, it requires longer signal clearance times.

4. The SPDI has a higher cost than the conventional or compressed diamond because of the need for a long, single-span structure and the need for retaining walls or reinforced earth walls along the mainline.

5. Where the mainline is over a crossroad, lighting is required under the structure.

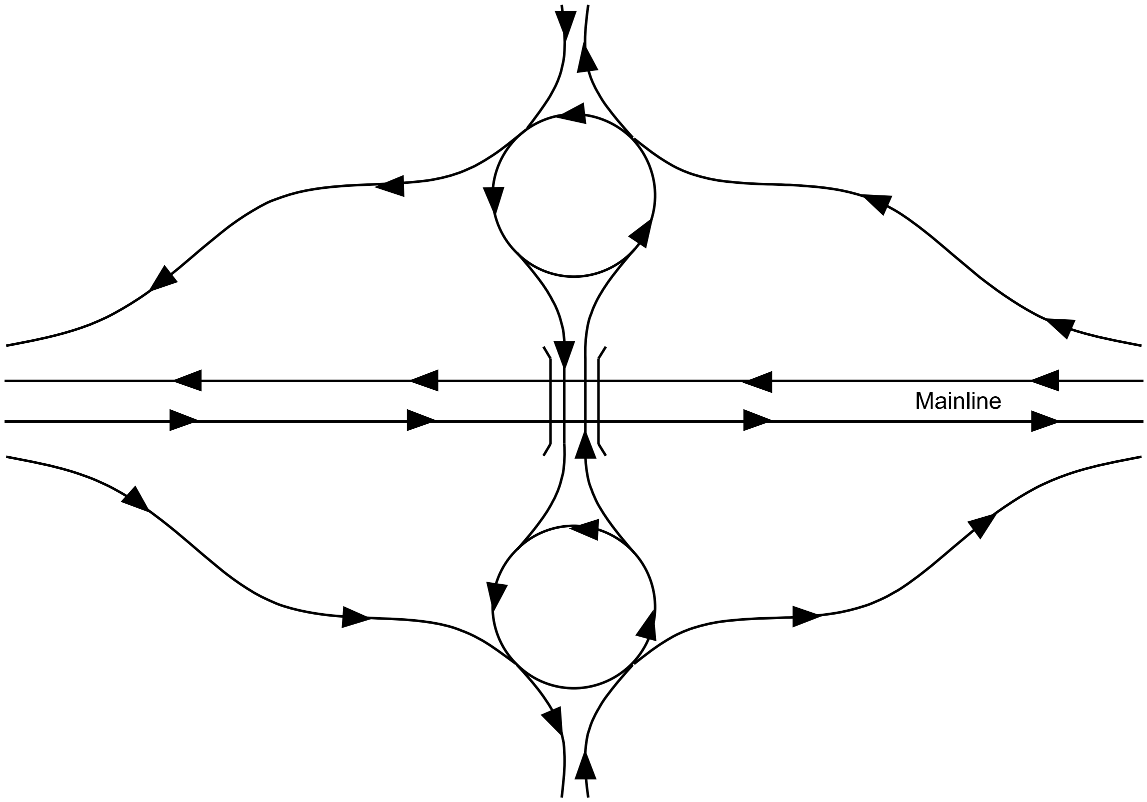
See NCHRP Report 345, *Single-Point Urban Interchange Design* and *Operational Analysis* for complete design details of a SPDI.

10.2.7 Double Roundabout Diamond

A double roundabout diamond, also called a dumbbell or dog-bone diamond interchange, contains roundabouts at each crossroad ramp. Figure 10.2-F presents a schematic of a double roundabout diamond interchange. Free-flow through movements are provided by using two-single or multi-lane roundabouts on the cross street to accommodate left and right turns and all movements on the cross street. The design provides a narrower bridge (no storage turn lanes) and the elimination of signal control at the interchange. Some of the advantages and disadvantages of the double roundabout diamond include:

Advantages

1. The roundabouts allow almost continuous flow; reduced delay for ramp traffic.



DOUBLE ROUNDABOUT DIAMOND INTERCHANGE
Figure 10.2-F

2. Signal coordination and progression issues are eliminated between the two ramp terminals.

3. The overpass bridge can be narrower, because left-turn lanes are eliminated from the bridge.

4. They are an easy-to-build step up from the diamond junction.

5. Reduces the number of conflict points.

Disadvantages

1. Profile grades may need to be 3 percent or flatter on all approaches to ensure adequate sight distance.

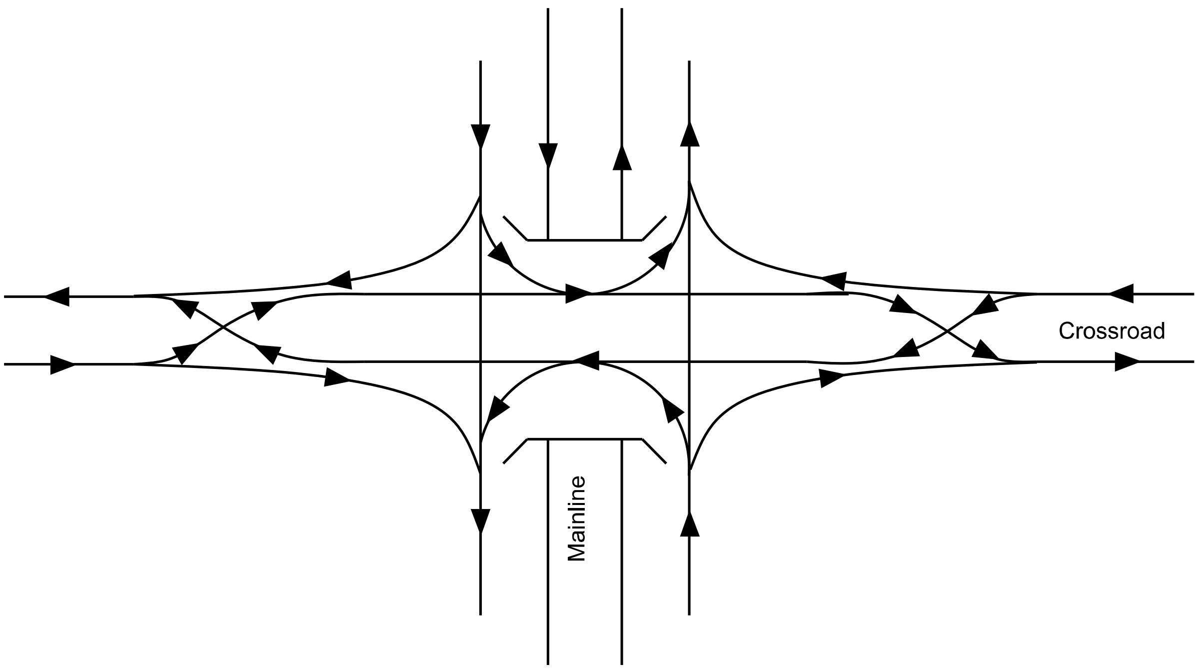
2. May not have sufficient capacity for high-volume intersections.

3. Pedestrian and bicycle movements may be hindered.

10.2.8 Diverging Diamond Interchange

The diverging diamond interchange transposes the opposing directions of crossroad traffic through the interchange, see Figure 10.2-G. This configuration allows all turning traffic from or onto the freeway to turn without crossing in front of the opposing traffic. By doing this, the two traffic signals only need to operate as a two-phase signal control instead of three-phase control for other diamond interchanges.

The primary reason for considering the diverging diamond interchange is due to its increased capacity and/or reduced lane requirements versus other diamond interchange configurations. It is only applicable in urban or suburban areas where operating speeds are no more than 40 to 50 miles per hour. Special consideration must be given to the design to ensure the interchange operates properly (e.g., flared approaches, channelization).



DIVERGING DIAMOND INTERCHANGE
Figure 10.2-G

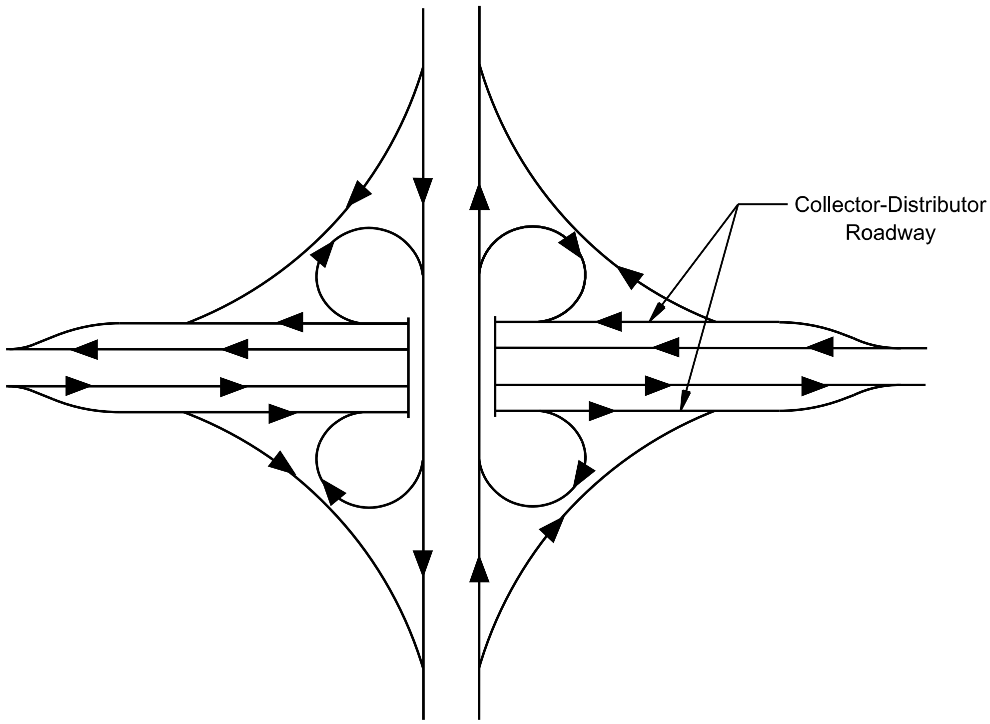
10.2.9 Full Cloverleafs

Cloverleaf interchanges are used at four-leg intersections and employ loop ramps to accommodate left-turn movements. Cloverleaf interchanges without collector-distributor (C-D) roadways are seldom considered a viable option. Full cloverleaf interchanges are those with loops in all four quadrants; all others are partial cloverleafs; see Section 10.2.10.

Where two access-controlled highways intersect, a full cloverleaf is the minimum type of interchange design that will suffice. In addition, they also may be used at the intersection of other multilane arterials to accommodate large volumes of traffic.

The operation of a cloverleaf with high weaving volumes is greatly improved through the addition of collector-distributor (C-D) roadways; see Section 10.3.8. The C-D roadways may be advantageous in suburban areas because of the need for smaller loops. This may reduce the amount of right of way acquisition necessary for the development of the interchange. Although right of way requirements may be reduced, overall costs usually increase due to longer and wider structures and additional pavement costs.

Figure 10.2-H provides a typical example of a full cloverleaf with C-D roads.



CLOVERLEAF INTERCHANGE WITH C-D ROADWAYS
Figure 10.2-H

Some of the advantages and disadvantages of full cloverleafs include:

<u>Advantages</u>

1. Full cloverleafs eliminate all vehicular stops through the use of free-flow terminals, and they provide continuous free-flow operation on both intersecting highways.

2. Full cloverleafs eliminate all at-grade intersections, eliminate left turns across traffic and, therefore, eliminate the need for traffic signals.

<u>Disadvantages</u>

1. Because of the geometric design of loops, full cloverleafs require large amounts of right of way.

2. They are typically more expensive than diamond interchanges due to considerably lengthier ramps, wider structures and, if provided, the additional cost of C-D roads.

3. The loops in cloverleafs result in a greater travel distance for left-turning vehicles than do diamonds, and the speeds on the ramps are generally slower.

4. Exit and entrance terminals are located before and after the crossroad structure, which require additional signing to guide motorists.

5. Weaving sections between loop ramps must be long enough to provide for satisfactory traffic operations.

6. Where the crossroad is an expressway or other multilane highway, a considerable length of access control distance is needed along the crossroad to the first point of access.

7. Pedestrian movements are difficult to accommodate.

Operational experience with full-cloverleaf interchanges has yielded several observations on their design. Subject to a detailed analysis on a site-by-site basis, the following generally characterize the design of cloverleafs:

1. Design Speed Impacts. For an increase in design speed, there will be an increase in travel distance and required right of way.

2. Loop Radii. Design of loop radii is highly dependent on the relative design speed of the two crossing roadways. Consistency with the exit speed on the upstream end and entrance speed requirements on the downstream end are essential.

3. Loop Geometry. Circular curve loop ramps are desirable geometrically because speeds and travel paths tend to be more constant and uniform. However, compound curves are often used as site conditions dictate. Transition of the design speed from curve to curve into and out of the loop is critical.

4. Loop Capacity. Expected design capacities for single-lane loops range from 800 to 1200 vehicles per hour. The higher volumes are generally only achievable where the design speed is 30 miles per hour or higher and few trucks use the loop.

5. Weaving Volumes. An auxiliary lane is typically provided between successive entrance/exit loops within the interior of a cloverleaf interchange. This produces a weaving section between the mainline and entering/exiting traffic. Where the total volume on the two successive ramps reaches approximately 1000 vehicles per hour, there may be a significant reduction of the through travel speed and level of service. Where this occurs, consider providing collector-distributor roadways.

6. Weaving Lengths. The minimum weaving length between the exit and entrance gores of loops on new cloverleaf interchanges without collector-distributor roadways or those undergoing major reconstruction should be at least 1000 feet or the distance determined by a capacity analysis, whichever is greater.

7. Collector-Distributor Roadways. The consideration of collector-distributor roadways should be an integral part of cloverleaf design. They deploy the exit in advance of the crossroad and encourage a lower speed weaving area, which is easier to match with the loop design.

10.2.10 Partial Cloverleafs

10.2.10.1 General

Partial cloverleaf interchanges are those with loops in one, two or three quadrants. See Figure 10.2-I. Several of the advantages and disadvantages for full cloverleafs also apply to partial cloverleafs (e.g., geometric restriction of loops). However, some specific advantages of partial cloverleafs include:

1. Partial cloverleafs provide access where one or more quadrants present adverse right of way and/or topographic problems that preclude a typical diamond interchange.

2. Partial cloverleafs may accommodate heavy left-turn traffic by means of a loop and thereby improve capacity, operations and safety.

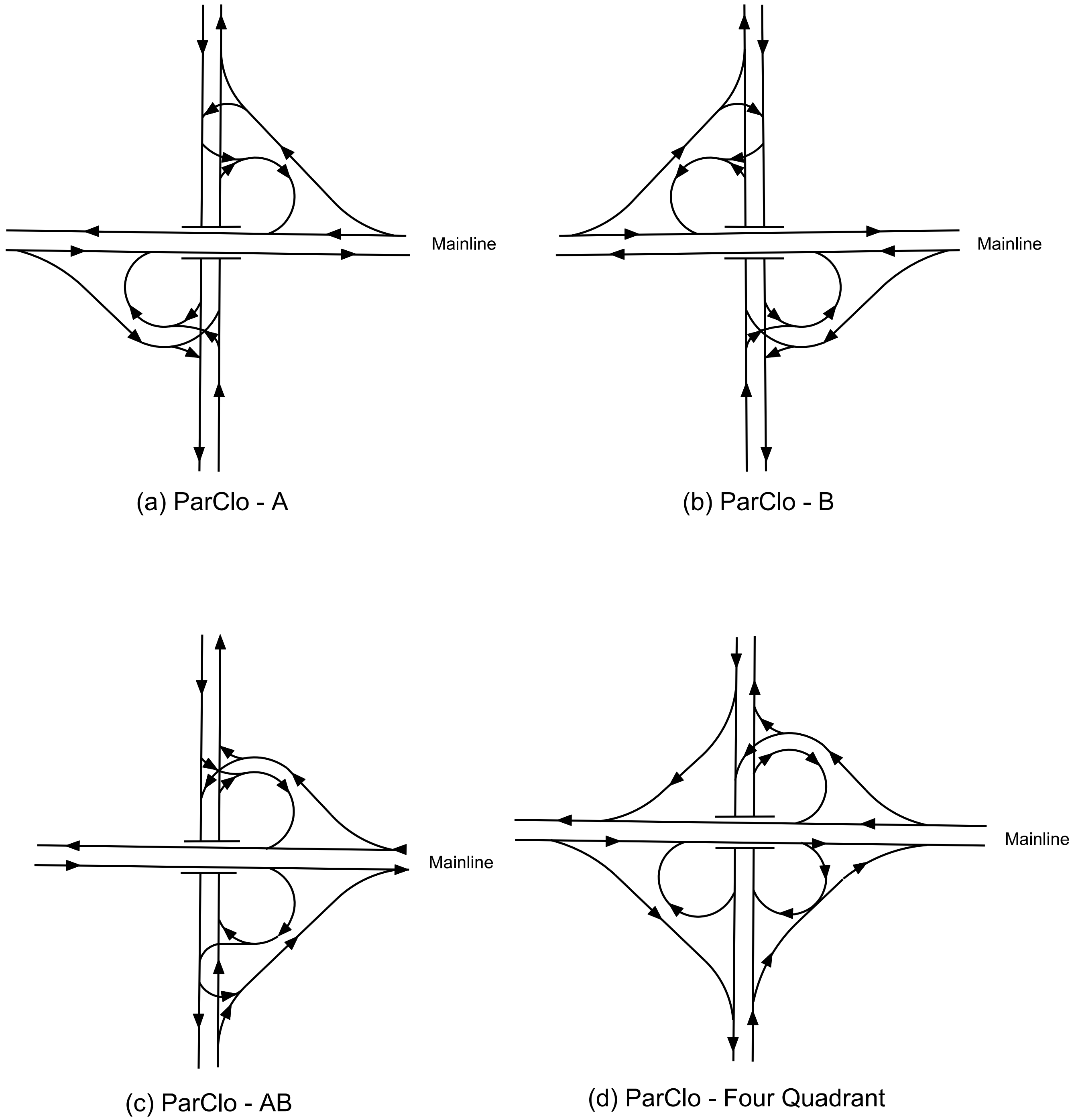
3. Depending upon site conditions, partial cloverleafs may offer the opportunity to eliminate or increase weaving distances.

10.2.10.2 ParClo-A Interchanges

The ParClo-A interchange is a high capacity service interchange appropriate in suburban areas. The two loop ramps are in opposite quadrants and accommodate the left-turning traffic from the cross street onto the freeway; see Figure 10.2-I(a). The term ParClo-A refers to the location of the loop ramps in relation to the driver approaching the interchange (cross street) is in advance of the cross street. This is true in either direction of travel on freeway. If signalized, the two intersections on the cross street are dependent on the radii of the loop ramps. With 150 feet radii loop ramps, the intersections are approximately 700 feet apart. These two-phase signalized intersections can be coordinated for efficient operation.

10.2.10.3 ParClo-B Interchanges

The ParClo-B interchange also has two loop ramps in opposite, but different quadrants than the ParClo-A interchange; see Figure 10.2-I(b). For the driver on the freeway approaching ParClo-B interchange, the loop ramp on the right side of the freeway is beyond the cross street. Drivers exiting the freeway make left turns through the loop ramps. The ParClo-B is appropriate in suburban areas where a freeway interchanges with an arterial street. Although it has similar capacity as the ParClo-A, it has different design and operational characteristics.



PARTIAL CLOVERLEAF INTERCHANGES
Figure 10.2-I

10.2.11 Three-Leg Interchanges

Three-leg interchanges are also known as trumpet (or jug handle), Y-type or T type interchanges. Figure 10.2-J illustrates examples of three-leg interchanges with different methods of providing the turning movements.

10.2.11.1 Trumpet (or Jug Handle)

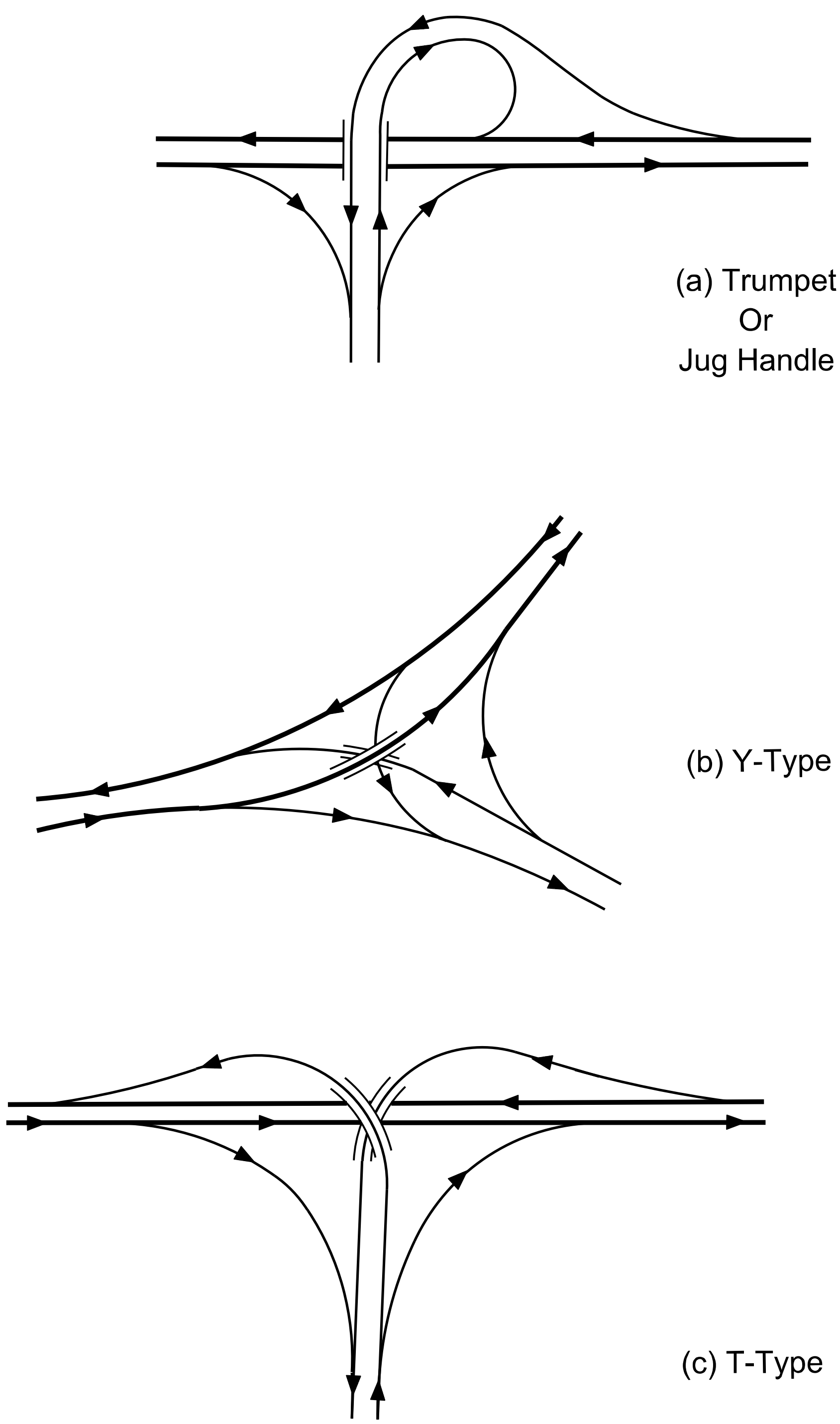
The trumpet (or jug handle) type is shown in Figure 10.2-J(a) where two of the turning movements are accommodated with direct connections, one with a semi-direct connection and one movement by a loop ramp. See Section 10.2.12 for a description of directional connection ramps. This most widely used three-leg interchange has a single (or twin) structure. The criteria for choosing the orientation of ramps depend on the expected traffic volumes or the left-turning movements. The semi-direct connection, sometimes called a jug handle, is used when the highest volume is the left-turning movement, and the lesser volume is carried by the lower capacity loop ramp. Semi-direct connections will handle traffic volumes in the range of 1200 to 1500 vehicles per hour, while loop ramps are limited to 800 to 1200 vehicles per hour. The trumpet interchange is often adaptable to the intersection of a major highway with a principal arterial or freeway.

10.2.11.2 Y-Type

The Y-Type is the costliest of three-leg interchanges, but highly efficient for traffic operations; see Figure 10.2-J(b). This interchange type has two or more structures or one three-level structure and provides high-capacity directional traffic movements (approximately 1200 to 1500 vehicles per hour per ramp) without loops. However, the use of this interchange can only be justified by the requirement to provide for directional traffic movements in excess of 800 vehicles per hour. This type of interchange is most common where two major roads join and continue as a single roadway. In addition, this type may be considered where right of way is limited.

10.2.11.3 T-Type

The T-type interchange is shown in Figure 10.2-J(c) where two of the turning movements are accommodated with direct connections and two with semi-direct connections. The criteria for choosing this type of interchange is the high volume of left-turn movements. Semi-direct connections are used when the highest volumes are left-turning movements. Semi-direct connections will handle traffic volumes in the range of 1200 to 1500 vehicles per hour, while loop ramps are limited to 800 to 1200 vehicles per hour. The T-type interchange is often adaptable to the intersection of a major highway with a principal arterial or freeway. Also, this type of interchange may be considered where the right of way is limited.



THREE-LEG INTERCHANGES
Figure 10.2-J

10.2.12 Directional Interchanges

The following definitions apply to directional interchanges:

1. Direct Connection. A ramp that does not deviate greatly from the intended direction of travel; see Figure 10.2-K(a), (b) and (c).

2. Semi-Direct Connection. A ramp where the driver first exits to the right, heading away from the intended direction of travel, gradually reversing, and then passing around other interchange ramps before entering the other road; see Figure 10.2-K(b) and (c).

Direct and semi-direct connections are used for heavy left-turn movements to reduce travel distance, increase travel speed and capacity, and eliminate weaving. These types of connections allow an interchange to operate at a better level of service than is possible with loops. The capacity of a direct or semi-direct connection is approximately 1200 to 1500 vehicles per hour per ramp lane. Exits and entrances on the left should be avoided.

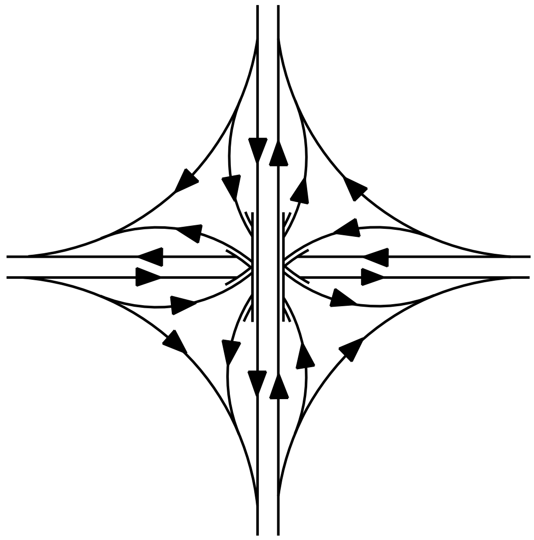
Directional interchanges are most often provided in urban or suburban areas at freeway-to-freeway or freeway-to-arterial intersections. In rural areas, there is generally an insufficient traffic volume to justify the use of direct or semi-direct connections in all quadrants. A directional interchange provides the highest possible capacity and level of service, but it is often costly to construct due to the number of structures required and amount of embankment. Because motorists perceive that higher operating speeds are possible on directional roadways, the alignment of these facilities should be as free flowing as practical.

10.2.13 Selection

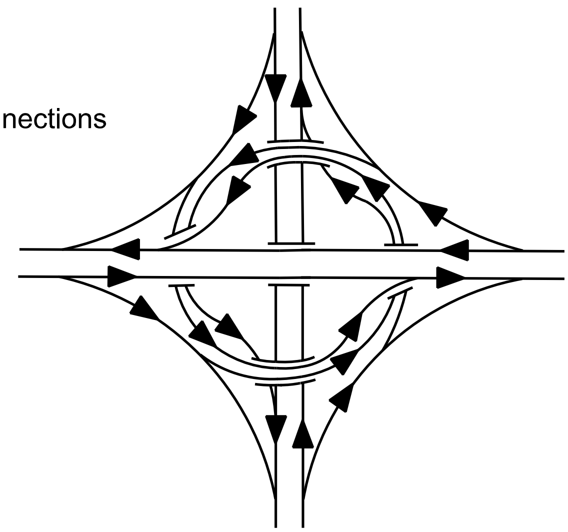
10.2.13.1 Evaluation Factors

The designer should evaluate the following factors when selecting an interchange type:

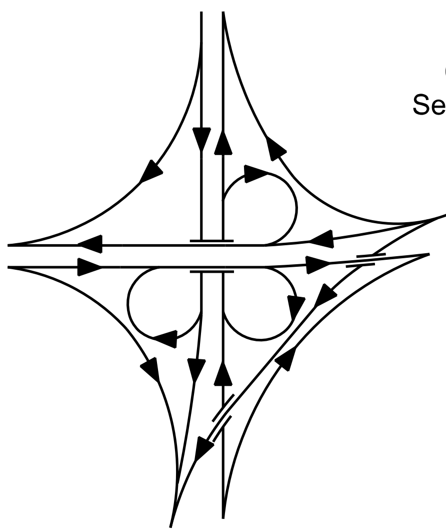
- compatibility with the highway system and functional classification of the intersecting highway;
- route continuity and uniformity with adjacent interchanges;
- level of service for each interchange element (e.g., freeway/ramp junction, ramp proper, ramp/crossroad terminal);
- operational and safety considerations (e.g., signing);
- availability of access control along the crossroad;
- road-user impacts (e.g., travel distance and time, convenience, comfort);
- constructability/maintenance of traffic;
- driver expectancy;
- topography and geometric design;
- right-of-way impacts and availability, construction and maintenance costs and potential for stage construction;
- accommodation of pedestrians and bicyclists on crossroad;
- environmental impacts; and
- potential growth of surrounding area.



(a) With Direct Connections



(b) With Direct and Semidirect Connections



(c) With Direct and Semidirect Connections and Loop Ramps

DIRECTIONAL INTERCHANGES
Figure 10.2-K

10.2.13.2 General Considerations

The designer should consider the following general factors that will influence the selection of an interchange type:

1. Basic Types. A freeway interchange will be one of two basic types. A systems interchange will connect a freeway to a freeway; a service interchange will connect a freeway to a lesser facility.

2. Freeways. For system interchanges of two fully access-controlled facilities, the minimum design will be a full cloverleaf interchange with collector-distributor roads. Where traffic volumes are significant, a directional interchange may be the most appropriate interchange type.

3. Movements. All interchanges should provide for all movements, even when the anticipated turning volume is low. An omitted maneuver may be a point of confusion to those drivers searching for the exit or entrance. In addition, unanticipated future developments may increase the demand for that maneuver.

4. Capacity. The need for loop ramps or other free-flowing ramps may depend upon the capacity of the ramp termini to adequately accommodate the turning traffic. Conduct a traffic analysis to determine if the ramp termini will be adequate and to determine the appropriate number of approach lanes on the crossroad and ramps.

5. Rural. In rural areas where interchanges occur relatively infrequently, the design can normally be selected strictly on the basis of service demand and analyzed as a separate unit. For most locations, the diamond or partial cloverleaf interchanges are the most appropriate interchange types.

6. Urban. In urban areas the selection of the interchange type is much more complex. In addition to the criteria above, the designer should consider the following factors:

    a. Right of Way. Right of way, in general, is more restricted in urban areas, thereby limiting the available interchange types. This may eliminate the use of a full cloverleaf. In highly restricted locations, the use of a tight diamond or single-point diamond interchange may be the only practical option.

    b. Spacing. Closely spaced interchanges may be influenced directly by the preceding or following interchange such that additional traffic lanes may be required to satisfy capacity, weaving and lane balance.

    c. High-Traffic Volumes. Ramps with high volumes may require free-flowing ramp crossroad terminals to adequately accommodate the turning traffic. High-traffic volumes may also cause problems with weaving sections. To accommodate these concerns may require partial cloverleafs.

    d. Urban System. Evaluate all interchanges along an urban route on a system-wide basis rather than on an individual basis. This will require a corridor analysis reviewing several alternative interchange layouts and types.

e. Crossroads. A thorough study of the crossroad is necessary to determine its potential for accommodating the increased volume of traffic that an interchange will discharge. The ability of the crossroad to receive and discharge traffic from the freeway has considerable bearing on the interchange geometrics (e.g., using loops to eliminate left-hand turns from a conventional diamond).

f. Environmental/Community Factors. Environmental concerns or community opposition to a particular interchange design may impact the selection of an interchange type. For example, a single-point diamond interchange or compressed diamond will require less right of way than a partial cloverleaf.

7. Turning Traffic. Where turning traffic is significant, the ramp profiles are best fitted when the major road is at the lower level. The ramp grades then assist turning vehicles to decelerate as they leave the major highway and to accelerate as they approach it. In addition, for diamond interchanges, the ramp terminal is visible to drivers as they leave the major highway.

10.2.13.3 Capacity (Traffic Volume) Considerations

Interchange type selection, in part, is based upon providing the capacity and level of service that is consistent with the type of highway (major vs. minor) and the anticipated traffic movement between the two facilities. In the hierarchy of interchanges, diamonds provide the lowest in traffic capacity followed in ascending order by partial cloverleafs, cloverleafs and directional. C-D roads can be used with all of these interchange types as necessary to enhance traffic flow and safety, and reduce weaving problems. They are particularly effective in urban freeway design where spacing between interchanges is less than the desired minimum. Figure 10.2-L provides capacity guidelines for initially determining the number of freeway lanes and type of interchange ramps. The final design must be checked with a traffic analysis.

10.2.13.4 Summary

The following presents a summary of the general application of the basic interchange types to general site conditions:

1. Diamond Interchanges. Where left-turning movements are low to and from the major highway, diamond interchanges are normally adequate. The capacity of diamond interchanges is limited by the capacity of the at-grade ramp/crossroad intersection. As left-turn movements at diamond interchanges increase, additional lanes and/or traffic signals at one or both ramp/crossroad intersections may be required.

2. Partial Cloverleaf Interchanges. Where one or more left-turn volumes are significant (500 vehicles per hour or more), loop ramp(s) may be added to provide the partial cloverleaf design to allow for continuous flow. Partial cloverleaf interchanges are an effective compromise between diamonds and full cloverleafs. Their usage may be dictated by limited right of way in one or more quadrants or by a need to maintain continuous flow on those left-turn movements that are disproportionately high or by operational limitations of the crossroad that require the elimination of certain crossing maneuvers.

3. Full Cloverleaf Interchanges with C-D roads. Full cloverleafs are the minimum type interchange used at the intersection of two fully access-controlled highways. Limiting factors in the selection of cloverleaf interchanges are the availability of large amounts of right of way (in the vicinity of 20 acres per quadrant) and the capacity for handling weaving at consecutive entrance/exit terminals. Collector-distributor roads may be necessary to eliminate the weaving problems along the mainline roadways.

4. Directional Interchanges. Interchanges with direct ramp or semi-direct ramp connections for all left-turning movements are often used at intersections of two high-volume freeways having large and nearly equal traffic volumes interchanging between the two facilities.

5. Other/Hybrid Interchanges. Other interchanges or combinations of the interchanges described above may be considered.

Segment(1)		**Traffic Volume (veh/h)**(1)	**Guidance**
Freeway(2)	Urban	0 – 5010	Two lanes in each direction
		5010 – 6680	Three lanes in each direction
		> 6680	Four lanes in each direction
	Rural		See *Highway Capacity Manual*.
Ramps		0 – 300	Single lane ramp
		300 – 500	Dual lefts OR free-flow rights
		500 – 800	Loop rams, semi-direct free flow
		800 -1200	Direct ramp, free flowing, no weaving
		> 1200	Two-lane entrance/exit ramps at freeway junction
Arterials		0 – 2220	Two lanes in each direction
		2220 – 3340	Three lanes in each direction

Notes: (1) All values are for general guidance. A traffic analysis is required for final design based on actual design conditions.

(2) Contact the SCDOT Planning Office for existing LOS for freeway segments.

INTERCHANGE CAPACITY GUIDELINES
Figure 10.2-L

10.3 INTERCHANGE SYSTEM OPERATIONS AND DESIGN

Part I "Roadway Design Elements" and Chapter 17 "Freeways" of the *SCDOT Roadway Design Manual* present Department criteria for several operational and geometric design elements (e.g., design speed, horizontal/vertical alignment, cross sections, frontage roads, over/under determinations, freeway lane drops) that apply to an interchange. This section discusses operational and design considerations that apply to the overall interchange system and, in many cases, are unique to interchange design.

10.3.1 <u>Interchange Spacing</u>

Interchange spacing is based on the demand for access, adequate distance to provide for signing and weaving and adequate distance to permit the adjacent interchanges to operate safely and efficiently for the appropriate level of service. Locations of adjacent interchanges must be evaluated to meet the appropriate weaving and signing criteria.

The desirable values usually allow adequate distances for an entering driver to adjust to the freeway environment, for proper weaving maneuvers between entrance and exit ramps and for adequate signing. However, considering the effects of existing streets and highways, traffic operations and environmental considerations, the spacing between adjacent interchanges may vary considerably. Minimum spacing should be 1 mile in urban areas and 3 miles in rural areas, based on crossroad to crossroad spacing. In urban areas, a spacing of less than 1 mile may be developed by using grade separated ramps and collector-distributor roads.

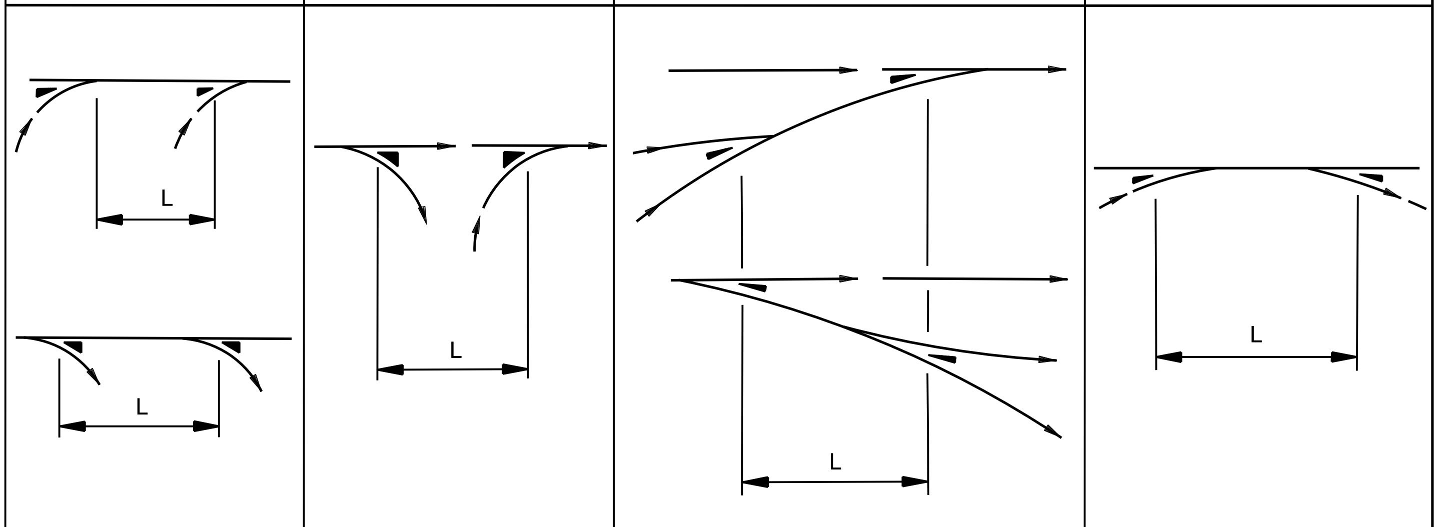
10.3.2 <u>Distance Between Successive Freeway/Ramp Junctions</u>

Successive freeway/ramp junctions may be placed relatively close to each other, especially in urban areas. The distance between the terminals should provide for vehicular maneuvering, signing and capacity. Figure 10.3-A provides recommended guidelines for spacing distances of various freeway/ramp junctions. The criteria in Figure 10.3-A should be considered for the initial planning stages of interchange location. The final decision on the spacing between freeway/ramp junctions should be based on the traffic analysis. Where the distance between the tapers of successive entrance and exit terminals is less than 1500 feet, consider connecting the two terminals with an auxiliary lane and provide a recovery area beyond the exit terminal.

10.3.3 <u>Basic Number of Lanes</u>

The basic number of lanes is the minimum number of lanes designated and maintained over a significant length of a route based on the overall operational needs and traffic volumes of that section. The number of lanes should remain constant over short distances. For example, do not drop a lane at the exit of a diamond interchange and then add it at the downstream entrance simply because the through traffic volume decreases between the exit and entrance ramps. Likewise, do not drop a basic lane between closely spaced interchanges simply because the estimated traffic volume does not warrant the higher number of lanes. Lane drops should only occur where there is an overall reduction in the traffic volumes on the freeway route as a whole.

RAMP TERMINAL SPACING GUIDELINES
Figure 10.3-A



EN-EN or EX-EX		EX-EN		Directional Ramps		EN-EX (Weaving)			
Full Freeway	CDR or FDR	Full Freeway	CDR or FDR	System Interchange	Service Interchange	System to Service Interchange		Service to Service Interchange	
						Full Fwy.	CDR or FDR	Full Fwy.	CDR or FDR
Minimum Lengths (L) Measured Between Successive Ramp Terminals									
1000 ft	800 ft	500 ft	400 ft	800 ft	600 ft	2000 ft	1600 ft	1600 ft	1000 ft

FDR - Freeway Distributor Road CDR - Collector-Distributor Road EN - Entrance EX - Exit

Note: The lengths are based on operational experience and the need for flexibility and adequate signing. They should be checked according to the procedure in the Highway Capacity Manual. The larger of the values is suggested for use. Also, a procedure for measuring the length of the weaving section is given in the Highway Capacity Manual.

10.3.4 Lane Balance

Lane balance is normally a major concern on high-volume urban freeways and a necessary element to realize efficient traffic operation through an interchange or series of interchanges. After the basic number of lanes is determined, the balance in the number of lanes should be checked on the basis of the following principles:

1. Exits. The number of approach lanes to the highway exit should equal the sum of the number of mainline lanes beyond the exit plus the number of exiting lanes minus one; see Figure 10.3-B. An exception to this principle would be at cloverleaf loop ramp exits that follow a loop ramp entrance or at exits between closely spaced interchanges (e.g., interchanges where the distance between the taper end of the entrance terminal and the beginning taper of the exit terminal is less than 1500 feet and a continuous auxiliary lane is used between the terminals). In these cases, the auxiliary lane may be dropped at a single-lane exit with the number of lanes on the approach roadway being equal to the number of through lanes beyond the exit plus the lane on the exit.

2. Entrances. At entrances, the number of lanes beyond the merging of the two traffic streams should be not less than the sum of the approaching lanes minus one; see Figure 10.3-B.

3. Lane Drops. Reduce the number of travel lanes on the freeway only one lane at a time.

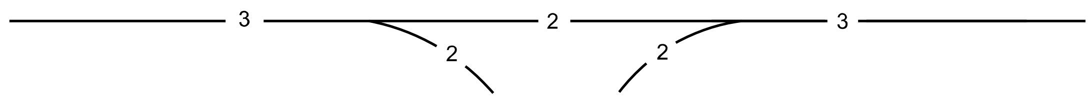
At exits, for example, dropping two mainline lanes at a two-lane exit ramp would violate the principle of lane balance. One lane should provide the option of remaining on the freeway. Lane balance would also prohibit immediately merging both lanes of a two-lane entrance ramp into a highway mainline without the addition of at least one additional lane beyond the entrance ramp. Figure 10.3-B illustrates how to coordinate lane balance and the basic number of lanes at an interchange. Figure 10.3-B also illustrates how to achieve lane balance at the merging and diverging points of branch connections.

10.3.5 Capacity and Level of Service

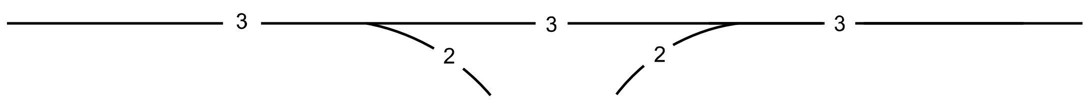
The capacity of an interchange will depend upon the operation of its individual elements which include:

- basic freeway section where interchanges are not present,
- freeway/ramp junctions,
- weaving areas,
- ramp proper,
- collector-distributor roadways, and
- ramp/crossroad intersections.

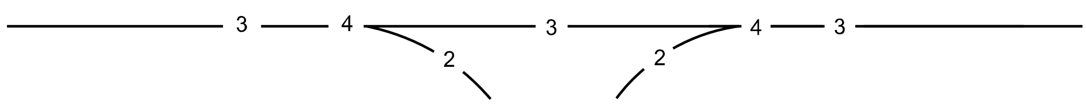
The basic capacity reference is the *Highway Capacity Manual* (HCM). The HCM and applicable software provide the analytical tools required to analyze the level of service for each element listed above. The design year for the interchange and crossroad will typically be the same as that for the freeway (i.e., 20 years).



(a) Lane Balance but no Compliance with Basic Number of Lanes



(b) No Lane Balance but Compliance with Basic Number of Lanes



(c) Compliance with Both Lane Balance and Basic Number of Lanes

Exit: $N_C = N_F + N_E - 1$

Entrance:

Max: $N_C = N_F + N_E$

Min: $N_C = N_F + N_E - 1$

Where:

N_C = Number of Lanes for Combined Traffic

N_F = Number of Lanes on Freeway

N_E = Number of Lanes on Exit or Entrance Ramp

(d) Lane Balance Equations

COORDINATION OF LANE BALANCE AND BASIC NUMBER OF LANES
Figure 10.3-B

Level of service values presented in Chapter 17 “Freeways” will also apply to interchanges. The level of service of each interchange element should be equal to the level of service provided on the basic freeway section. Individual elements should not operate at more than one level of service below that of the basic freeway section. In addition, the operation of the ramp/crossroad intersection in urban areas should not impair the operation of the mainline. This will likely involve a consideration of the operational characteristics on the minor road for some distance in either direction from the interchange.

10.3.6 Auxiliary Lanes

As applied to interchange design, auxiliary lanes are most often used to comply with the principle of lane balance, to increase capacity, to accommodate weaving or to accommodate entering and exiting vehicles. The operational efficiency of the freeway may be improved if a continuous auxiliary lane is provided between entrance and exit terminals where interchanges are closely spaced. An auxiliary lane may be dropped at an exit if properly signed and designed. The following statements apply to the use of an auxiliary lane within or between interchanges:

1. Within Interchange. Figure 10.3-C provides the basic schematics of alternative designs for adding and dropping auxiliary lanes within interchanges. The selected design will depend upon traffic volumes for the exiting, entering and through movements.

2. Between Interchanges. Where interchanges are closely spaced, the designer should provide an auxiliary lane where the distance between the end of the entrance terminal and beginning of the exit taper is less than 1500 feet.

Design details for exit and entrance ramps are provided in Section 10.4.1 and Section 10.4.2. The design details for freeway lane drops are provided in Section 17.5.1.

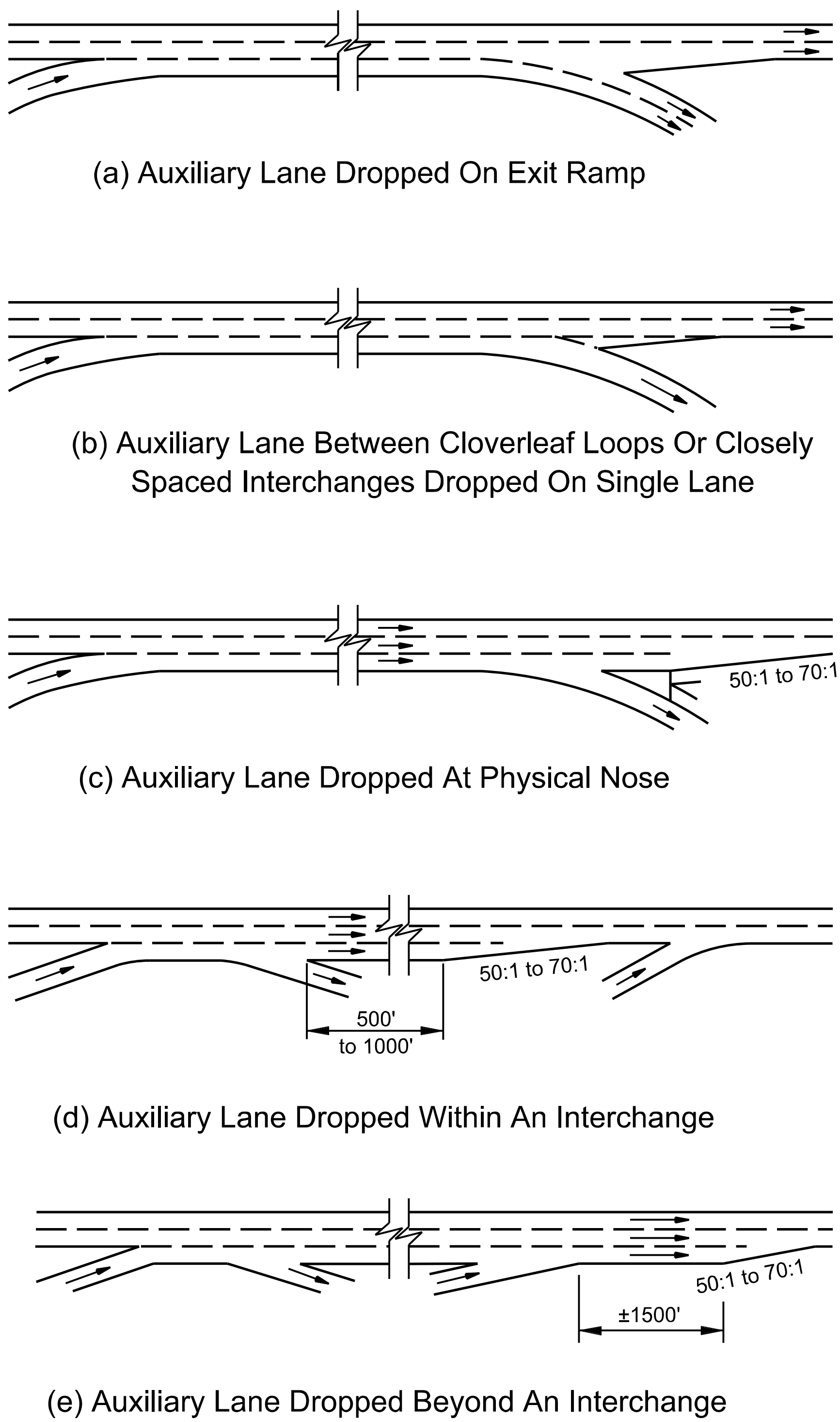
10.3.7 Weaving Sections

Weaving sections are highway segments where the pattern of traffic entering and exiting at contiguous points of access results in vehicular paths crossing each other. The turbulent effect of weaving operations can result in reduced operating speeds and levels of service for the through traffic. Weaving sections may be eliminated at an interchange between two major highways by using direct or semi-direct connections or by using collector-distributor roadways.

Consider the following for weaving sections:

1. Weave Length. Weaving sections on freeways other than cloverleafs should be at least 1000 feet or the length determined using the *Highway Capacity Manual* (HCM), whichever is greater.

2. Level of Service. The level of service of a weaving section should be the same as that for the adjacent mainline; however, at a minimum, it can be one level lower. A higher volume in weaving sections may be accommodated and their adverse impact on through traffic minimized by providing the weaving section on collector-distributor roadways. Section 10.3.8 discusses the use and design of collector-distributor roadways.



(a) Auxiliary Lane Dropped On Exit Ramp

(b) Auxiliary Lane Between Cloverleaf Loops Or Closely Spaced Interchanges Dropped On Single Lane

(c) Auxiliary Lane Dropped At Physical Nose

(d) Auxiliary Lane Dropped Within An Interchange

(e) Auxiliary Lane Dropped Beyond An Interchange

AUXILIARY LANES WITHIN AN INTERCHANGE
Figure 10.3-C

10.3.8 Collector-Distributor Roadways

10.3.8.1 Usage

A collector-distributor (C-D) roadway is an auxiliary roadway parallel to and separated from the main traveled way that serves to collect and distribute traffic from multiple access points. It provides greater capacity and permits higher operating speeds to be maintained on the main traveled way. C-D roadways may be provided at single interchanges, through two adjacent interchanges or, in urban areas, continuously through several interchanges. Figure 10.2-H illustrates a schematic of a C-D roadway within a full cloverleaf interchange.

Usually, interchanges designed with single exits are superior to those with two exits, especially if one exit is a loop ramp or the second exit is a loop ramp preceded by a loop entrance ramp. Whether used in conjunction with a full cloverleaf or with a partial cloverleaf interchange, the single-exit design may improve the operational efficiency of the entire interchange. C-D roadways use the single exit approach to improve the interchange operational characteristics. C-D roadways will:

- remove weaving maneuvers from the mainline and transfer them to the slower speed C-D roadways,
- provide high-speed single exits and entrances from and onto the mainline,
- satisfy driver expectancy by placing the exit before the grade separation structure,
- simplify signing and the driver decision-making process, and
- provide uniformity of exit patterns.

C-D roadways are most often warranted when traffic volumes (especially in weaving sections) are so high that the interchange cannot operate at an acceptable level of service. They also may be warranted where the speed differential between weaving and non-weaving vehicles is significant.

10.3.8.2 Design

When designing C-D roadways, consider the following:

1. Design Speed. The design speed of a C-D roadway usually ranges from 40 to 65 miles per hour. Typically, use a design speed within 10 miles per hour of the mainline design speed.

2. Lane Balance. Maintain lane balance at the exit and entrance points of the C-D roadways; see Section 10.3.4.

3. Width. C-D roadways may be one or two lanes, depending upon the traffic volumes and weaving conditions. C-D roadways are typically designed similar to ramps with traveled way widths of either 16 feet (1 lane) or 24 feet (2 lanes).

4. Separations. The separation between the C-D roadway and mainline should be as wide as practical. At a minimum, the separation should allow for shoulder widths equal to that on the mainline and for a suitable barrier to prevent indiscriminate crossovers.

5. Terminal Designs. Section 10.4 discusses the design of freeway/ramp junctions. These criteria also apply to C-D roadway/ramp junctions.

10.3.9 Route Continuity

The major route should flow continuously through an interchange. For freeway and expressway routes that change direction, the driver should not be required to change lanes or exit to remain on the major route. Route continuity without a change in the basic number of lanes is consistent with driver expectancy, simplifies signing and reduces the decision demands on the driver. Interchange configurations should not necessarily favor the heavier traffic movement. There may occasionally be sites where it is advisable to design the interchange to provide route continuity despite the traffic volume movements.

10.3.10 Uniformity

Interchange configurations along a route should be uniform from one interchange to another. All ramps should exit and enter on the right except under highly unusual conditions. Dissimilar arrangements between interchanges can cause confusion resulting in undesirable lane switches, reduced speeds, etc., especially in urban areas where interchanges are closely spaced.

10.3.11 Signing and Marking

Proper interchange operations depend on the compatibility between its geometric design and the traffic control devices at the interchange. The proper application of signs and pavement markings will increase the clarity of paths to be followed, safety and operational efficiency. The logistics of signing along a highway segment will also influence the minimum acceptable spacing between adjacent interchanges. The *MUTCD* provides guidelines and criteria for the placement of traffic control devices at interchanges.

10.3.12 Ramp Metering

Ramp metering may be used to improve freeway operations. Ramp metering consists of signals installed on entrance ramps before the entrance terminal to control the number of vehicles entering the freeway. The traffic designer will determine the need for ramp metering. If used, the roadway designer will need to coordinate with the traffic designer to determine the placement of the ramp signal to ensure that there is a sufficient storage area before the ramp signal and that sufficient acceleration distance is available beyond the signal to allow a vehicle to reach freeway speed.

10.3.13 Grading and Landscaping

Consider the grading around an interchange early in the design process. Alignment, fill-and-cut sections, median widths, lane widths, drainage, structural design and infield contour grading all affect the aesthetics of the interchange. Properly graded interchanges allow the overpassing structure to blend naturally into the terrain. In addition, ensure that the crossroad and ramp slopes are not so steep that they compromise safety and can support plantings to prevent erosion and enhance the appearance of the area. Flatter slopes also allow easier maintenance. Transitional grading between cut-and-fill slopes should be long and natural in appearance. The designer must ensure that plantings will not affect the sight distance within the interchange and that larger plantings are a significant distance from the traveled way.

Include a contour grade detail for interchanges in the plans.

10.3.14 Operational/Safety Considerations

Operations and safety are important considerations in interchange design. The following summarizes several general factors:

1. Exit Ramps. For exit ramps, consider the following:

    a. Signing. Proper advance signing of exits is essential to allow necessary lane changes before the exit.

    b. Deceleration. Provide sufficient distance to allow safe deceleration from the freeway design speed to the design speed of the first governing geometric feature on the ramp, typically a horizontal curve.

2. Entrance Ramps. Provide an acceleration distance of sufficient length to allow a vehicle to attain an appropriate speed for merging. Where entrance ramps enter the mainline on an upgrade, the acceleration distance may need to be lengthened, or an auxiliary lane may be required to allow vehicles to reach a safe speed prior to merging.

3. Driver Expectancy. Ensure that the interchange is designed to conform to the principles of driver expectancy. These may include the following:

    a. Left-Hand Exits and Entrances. Avoid left-hand exit or entrance terminals. Drivers expect single-lane exit and entrance terminals to be located on the right side of the freeway.

    b. Horizontal Alignment. Do not locate exit ramps so that it gives the appearance of a continuing mainline tangent as the mainline curves to the left.

    c. Consistency. Do not mix operational patterns between interchanges, lane continuity or interchange types.

    d. Lane Balance. Provide lane balance and the basic number of lanes on the freeway.

    e. Spacing. Provide sufficient spacing between interchanges to allow proper signing distances to decision points.

4. Roadside Safety. Because of the typical design features at interchanges, many fixed objects may be located within interchanges (e.g., signs at exit gores, bridge piers, rails). Avoid locating these objects near decision points, make them breakaway or shield them

with barriers or impact attenuators. See the AASHTO *Roadside Design Guide* for detailed guidance on roadside safety.

5. Traffic-Controlled Ramp Terminals. The designer must ensure that the ramp/crossroad intersection has sufficient capacity so that the queuing traffic at the crossroad intersection does not backup onto the freeway. Also, sufficient access control and intersection sight distance must be maintained along the crossroad to allow the ramp intersection to work properly. Provide sufficient sight distance to the ramp/crossroad traffic control devices.

6. Wrong-Way Maneuvers. Provide channelized medians, islands and/or adequate signing to minimize wrong-way possibilities. Avoid designs that may result in poor visibility, confusing ramp arrangements or inadequate signing.

7. Pedestrians and Bicyclists. Use signing and lane markings to increase awareness of pedestrians and bicyclists. Signing, crosswalks, barriers, over and underpasses, bridge sidewalks and other traffic control devices may be required to manage traffic movements and to control pedestrian and bicycle movements.

10.3.15 Geometric Design Criteria

Design all roadways through an interchange with the same criteria as used for the approaches including design speed, sight distance, horizontal and vertical alignment, cross section and roadside safety elements. In addition, consider the following:

1. Design Year. Typically, use a 20-year design period based on the anticipated project letting date.

2. Design Speed (Crossroad). The crossroad design speed should be based on the functional classification and urban or rural classification; see the geometric design tables in Chapters 14 “Local Roads and Streets,” Chapter 15 “Collector Roads and Streets” and Chapter 16 “Rural and Urban Arterials.”

3. Horizontal Alignment. In general, design the alignment of the freeway and crossroad through the interchange on a tangent. Where this is not practical, consider the following:

    a. Freeway Mainline. Minimize locating exit terminals where the freeway mainline curves to the left. If it cannot be avoided, provide an abrupt exit taper.

    b. Freeway/Ramp Junctions. Design the freeway alignment so that only one exit terminal departs from the mainline curving to the right, or design the mainline curve to lie entirely within the limits of the interchange and away from the exit and entrance terminals.

    c. Superelevation. Desirably, design the horizontal alignment so that superelevation and superelevation transitions will not be required through the freeway/ramp junctions or through the ramp/crossroad intersection.

    d. Crossroad. Where a curve is necessary, provide a significantly large horizontal curve so that superelevation is not required on the crossroad, if practical.

4. Vertical Alignment. Vertical profiles for both roadways through the interchange should be as flat as practical. Where compromises are necessary, use the flatter grade on the major facility. In addition, the designer should consider the following:

   a. Sight Distance. To improve the sight distance to exit gores, locate exit ramp terminals and major divergences where the mainline is on an upgrade.

   b. Ramps. Ramps should depart from the mainline where there will be no vertical curvature to restrict visibility along the ramp. Avoid ramp designs that drop out of sight. Also, provide flat approach grades adjacent to the crossroad. For additional information on storage platforms at the ramp/crossroad intersection, see Section 9.2.7.

   c. Exit Ramp Terminals. Where a freeway is proposed to cross over the crossroad, locate the exit ramp terminals on the mainline no closer than 1000 feet from the high point of a crest vertical curve on the mainline. This will ensure that no hidden ramps exist and will provide for safer operations at the exit ramp terminal.

   d. Turning Trucks. Large trucks may become unstable when executing a nonstop, left turn from a crossroad on a downgrade. The combination of a downgrade, sharp turning maneuvers onto a ramp and reverse superelevation may produce instability in large trucks. Therefore, the maximum grade for all crossroads associated with these conditions is desirably 2 percent through the ramp/crossroad terminal. For existing crossroads to remain in place, limit the downgrade to 3 percent. At a maximum, limit the up and downgrades to 4 percent.

5. Sight Distance. Because of the additional demand placed on the driver at an interchange, the designer should consider the following sight distance elements:

   a. Stopping Sight Distance. Provide adequate stopping sight distance on both intersecting highways throughout the interchange and on all ramps. Check both the vertical and horizontal alignment to ensure that the location of piers, abutments, structures, bridge rails, vertical curves, etc., will not restrict sight distance. Chapter 5 “Horizontal Alignment” discusses the application of horizontal sight distance. Chapter 6 “Vertical Alignment” discusses the application of vertical sight distance.

   b. Decision Sight Distance. Desirably, provide decision sight distance to all decision points (e.g., exit and entrance terminals); see Section 4.3. Desirably, at exit ramps, use the pavement surface for the height of object (i.e., 0.0 inches). Driver expectancy should not be violated.

   c. Intersection Sight Distance. Section 4.4 discusses intersection sight distance (ISD), which is also applicable at ramp/crossroad intersections (non-merging sites). Section 10.6 provides additional ISD guidance that should be considered at ramp/crossroad intersections that are stop controlled.

6. Ramp/Crossroad Intersections. When designing the ramp/crossroad intersection, consider the following:

a. Angle of Ramp Intersection. To determine the appropriate angle for the ramp/crossroad intersection, see Section 9.2.6.

b. Access Control. To determine the required length of access control along the crossroad at the interchange, see Section 3.8.

c. Left-Turn Lanes. Select the appropriate left-turn lane lengths based on the design speed of the crossroad and/or the required storage lengths; see Section 9.5.

d. Turning Movements. Check the ramp/crossroad intersection with the applicable design vehicle turning template or use a computer-simulated turning template program. Use the WB-62 design vehicle for determining turning radii, curb locations, median noses, etc., at the ramp/ crossroad intersection. Use the WB-67 design vehicle for determining storage lengths (e.g., left-turn lanes), median widths, etc., at the ramp/crossroad intersection.

7. Mainline/Crossroad Point of Intersection. Once Items 1 through 6 above have been determined, the designer must decide where the mainline alignment best intersects with the crossroad. The overall size of the interchange, crossroad grade lines, required length of access control along the crossroad, access to property at the ends of access control on the crossroad, and topography are the most influential factors in this determination. Complete this investigation before the detailed design of an interchange is initiated.

10.3.16 Reviewing for Ease of Operation

The designer should review the proposed design from the driver's perspective. This involves tracing all possible movements that an unfamiliar motorist would drive through the interchange. Review the plans for areas of possible confusion, proper signing, ease of operation and to determine if sufficient weaving distance and sight distances are available. Review both day and nighttime operations. Consider the peak-hour volumes, number of traffic lanes, etc., to determine the type of traffic the driver may encounter.

10.4 FREEWAY/RAMP JUNCTIONS

10.4.1 Exit Ramps

10.4.1.1 Types of Exit Ramps

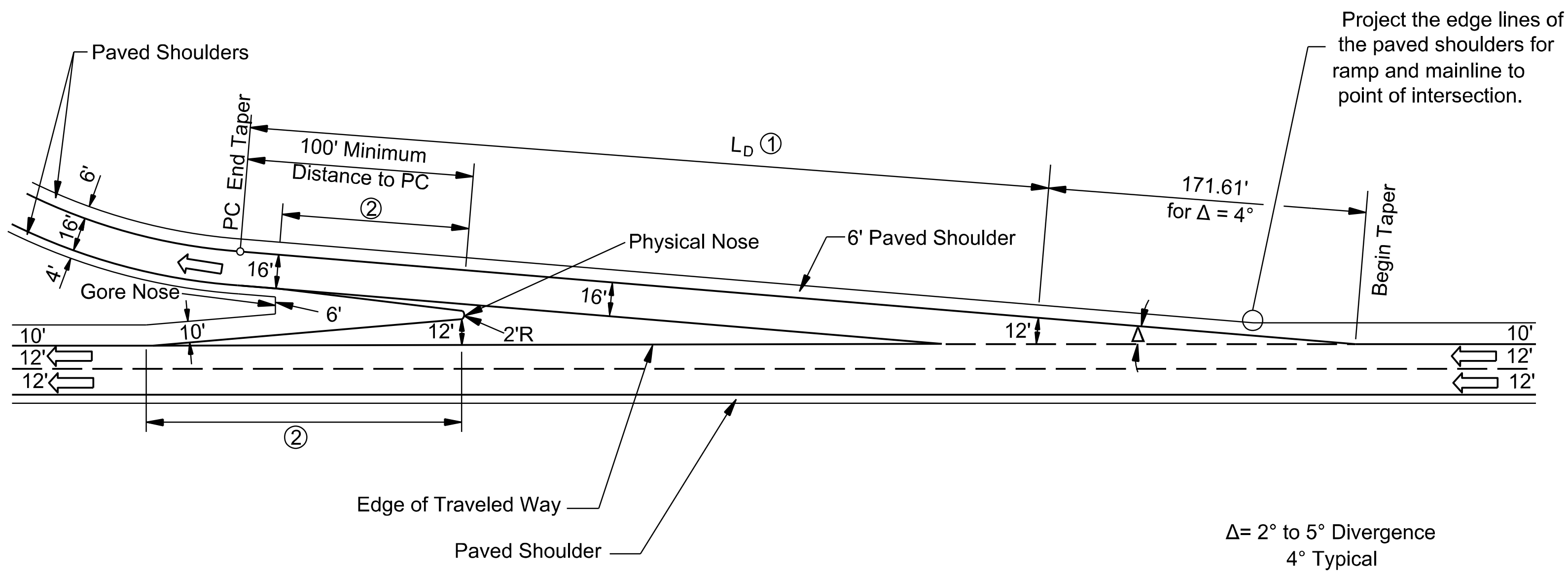
There are two basic types of exit freeway/ramp junctions — taper design and parallel design. Figures 10.4-A, 10.4-B and 10.4-C illustrate these designs. For most new and reconstructed ramps, SCDOT prefers to use the taper design. However, the designer may consider using the parallel design where:

- a ramp exit is just beyond an overpass structure and there is insufficient sight distance available to the ramp gore;
- the need is satisfied for a continuous auxiliary lane (see Section 10.3.6);
- the geometrics of the exit ramp are such that the taper design cannot satisfy the length needed for deceleration; or
- the exit ramp departs from a horizontal curve on the mainline. In this case, the parallel design is less confusing to through traffic and will normally result in smoother operations. It is also easier to design the superelevation transition with a parallel design.

10.4.1.2 Taper Rates

The taper rate applies to the rate at which the ramp diverges from the mainline. The following taper rates apply:

1. Taper Exit Design. The taper angle can vary between 2 and 5 degrees. For the typical SCDOT ramp design, the divergence angle is 4 degrees as illustrated in Figure 10.4-A.
2. Parallel Exit Design. The taper rate applies to the beginning of the parallel lane. This distance is typically 300 feet (i.e., 25:1) as illustrated in Figures 10.4-B and 10.4-C.

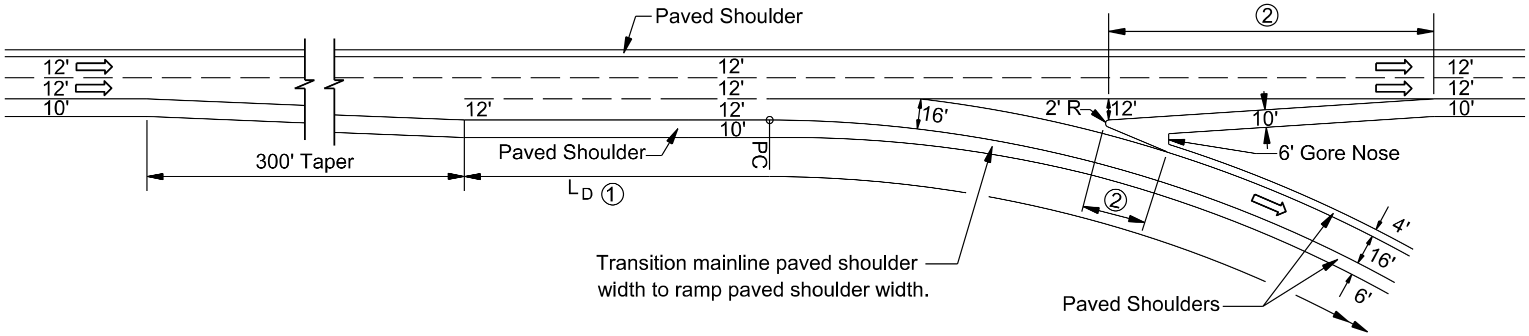


TAPER EXIT RAMP
Figure 10.4-A

Notes: 1. L_D is the deceleration distance required for a vehicle to slow down from the mainline design speed to the design speed of the first geometric control on the the ramp; see Section 10.4.1.3.

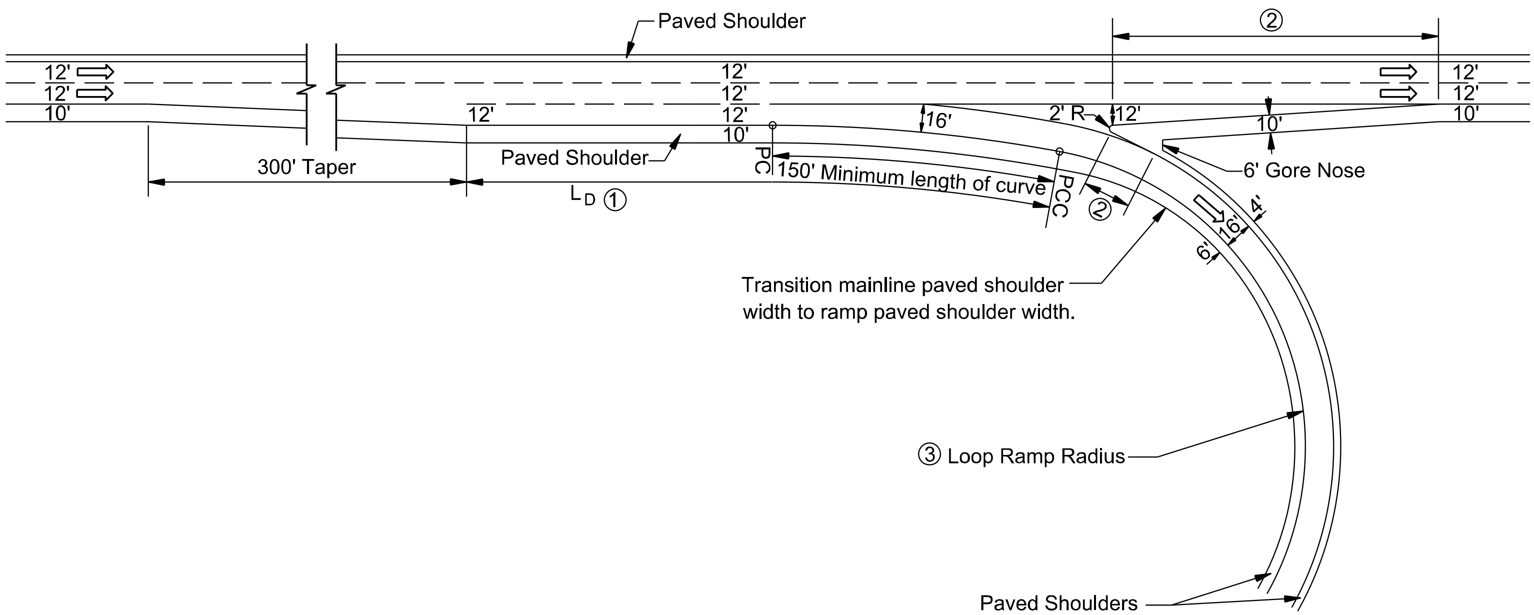
2. See Figure 10.4-G for the taper rate (z) to transition the offset physical nose to the normal traveled way width.

PARALLEL EXIT RAMP
Figure 10.4-B



Notes: 1. L_D is the deceleration distance required for a vehicle to slow down from the mainline design speed to the first geometric control on the ramp. See Section 10.4.1.3.

2. See Figure 10.4-G for the taper rate (z) to transition the offset physical nose to the normal traveled way width.



PARALLEL EXIT FOR LOOP RAMP
Figure 10.4-C

Notes: 1. L_D is the deceleration distance required for a vehicle to slow down from the mainline design speed to the first geometric control on the ramp. See Section 10.4.1.3.

2. See Figure 10.4-G for the taper rate (z) to transition the offset physical nose to the normal traveled way width.

3. Offset Loop Ramp Radius 24 feet from outside edge off mainline travel way.

10.4.1.3 Deceleration

Sufficient deceleration is needed to safely and comfortably allow an exiting vehicle to depart from the mainline. The following will apply:

1. Taper Exit. All deceleration should occur within the full width of the deceleration lane. The length of deceleration will depend upon the design speed of the mainline and design speed of the first governing geometric control on the ramp, typically a horizontal curve. This distance is measured from where the ramp becomes 12 feet wide to the first geometric control.

2. Parallel-Lane Exit. Design the departure curve or taper based on the design speed of the roadway being departed. The deceleration length begins where the full width of the parallel lane becomes available and ends where the departure curve of the ramp begins; see Figures 10.4-B and 10.4-C.

Figure 10.4-D provides the deceleration distances for various combinations of highway design speeds and ramp design speeds. If the deceleration distance is on a downgrade of 3 percent or more, adjust the deceleration distance according to the criteria in Figure 10.4-E.

* * * * * * * * *

Example 10.4-1

Given:	Highway Design Speed	=	70 miles per hour
	First Exit Curve Design Speed	=	45 miles per hour
	Average Grade	=	5 percent downgrade

Problem: Determine length of deceleration required.

Solution: Figure 10.4-D yields a minimum deceleration length of 390 feet on the level. According to Figure 10.4-E, this should be increased by 1.35.

Therefore: L_D = (390)(1.35)
L_D = 527 feet

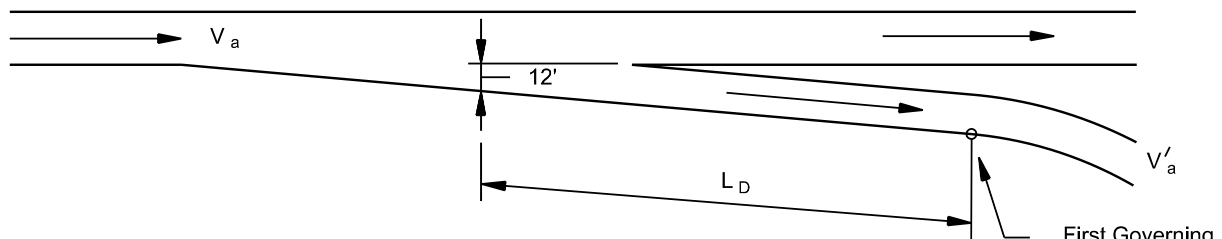
Provide a 530-foot deceleration length from the full width of the exit lane to the PC of the first exit curve.

* * * * * * * * *

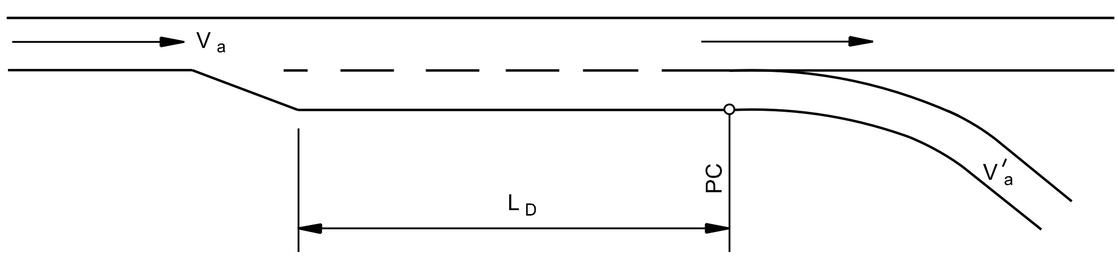
10.4.1.4 Sight Distance

Desirably, provide decision sight distance approaching a freeway exit. At a minimum, this sight distance should exceed the stopping sight distance by 25 percent. This sight distance should be available throughout the freeway/ramp junction (e.g., from the beginning taper to the gore nose; see Figures 10.4-A and 10.4-B). This sight distance is particularly important for exit loops immediately beyond a structure. Vertical curvature or bridge piers can obstruct the exit points if not carefully designed. The desirable height of object will be 0.0 feet (the roadway surface); however, it is acceptable to use 2 feet.

Design Speed of Highway (mph)	Speed Reached (mph)	L_D = Length of Deceleration (ft)								
		For Design Speed of First Governing Geometric Control (mph)								
		Stop	15	20	25	30	35	40	45	50
		For Average Running Speed (mph) (V'_a)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	—	—	—	—	—
35	32	280	250	210	185	150	—	—	—	—
40	36	320	295	265	235	185	155	—	—	—
45	40	385	350	325	295	250	220	—	—	—
50	44	435	405	385	355	315	285	225	175	—
55	48	480	455	440	410	380	350	285	235	—
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390



Taper Design



Parallel Design

Notes:

1. *The deceleration lengths are calculated from the distance needed for a passenger car to decelerate from the average running speed of the highway mainline to the average running speed of the first governing geometric control.*
2. *These values are for grades less than 3 percent. See Figure 10.4-E for steeper downgrades.*
3. *Select the actual design speed of the mainline and ramp when using this figure.*

LENGTH FOR DECELERATION
Figure 10.4-D

Direction of Grade	**Ratio of Deceleration Length on Grade (G) to Length on Level**		
	G < 3%	**3% ≤ G < 5%**	**5% ≤ G < 6%**
Downgrade	1.0	1.2	1.35
Upgrade	1.0	0.9	0.8

Notes: 1. *Figure applies to all highway design speeds.*

2. *The grade in the table is the average grade over the distance used for measuring the length of deceleration. See Figures 10.4-A and 10.4-B.*

GRADE ADJUSTMENTS FOR DECELERATION LENGTHS
Figure 10.4-E

10.4.1.5 Horizontal Alignment

Develop the superelevation for horizontal curves at the freeway/ramp junction based on the principles of superelevation as discussed in Section 5.3 for mainline highways. In addition, the following criteria are applicable to superelevation development at freeway ramp/junction exits:

1. Design Speed. Desirably, the design speed of the mainline roadway will be used as the design speed for any horizontal curves at the freeway/ramp exit. As discussed in Section 10.4.1.3, the freeway/ramp exit should provide sufficient distance for a vehicle to decelerate from the mainline design speed to the design speed of the first controlling design element of the exit ramp. This could be a horizontal curve in the vicinity of the exit gore. If the necessary deceleration distance is available, the design speed of the horizontal curve at a minimum may be equal to the design speed of the ramp proper; see Section 10.5.

2. Maximum Superelevation. The e_{max} that is applicable to the mainline (see Section 5.3) will also apply to horizontal curves at the freeway/ramp exit. In most cases, this will be e_{max} = 8.0 percent.

3. Minimum Curve Radius. Use the applicable e_{max} figure in Section 5.3 to determine the minimum radius for horizontal curves at freeway/ramp exits. The designer will use the selected design speed and appropriate design superelevation (e_d) to determine the minimum radius.

4. Transition Length. The designer must transition the exit ramp cross slope on tangent (typically 2.00 percent) to the superelevation rate for the horizontal curve. The following applies:

    a. The transition should not begin until the exit ramp has reached a minimum 12-foot width.

    b. The maximum relative gradient should not exceed the criteria in Figure 5.3-A. The relative gradient is measured between the outside edge of ramp traveled way and the inside edge of ramp traveled way.

c. If practical, approximately 67 percent of the transition length should be on the tangent and approximately 33 percent on the curve.

5. Point of Revolution. The designer may choose to put the point of revolution on either the inside or outside of the ramp traveled way.

10.4.1.6 Cross Slope Rollover

The cross slope rollover is the algebraic difference between the transverse slope of the through lane and the transverse slope of the exit lane and/or gore. The following will apply:

1. Up to Physical Nose. The cross slope rollover should not exceed 5 percent.
2. From Physical Nose to Gore Nose. The cross slope rollover should not exceed 7 percent.
3. Drainage Inlets. Where required, these are normally placed between the physical gore and gore nose. The presence of drainage inlets may require two breaks in the gore cross slope. These breaks should meet the criteria in Items 1 or 2 above, depending on the inlet location.

See Section 10.4.1.8 for gore nose definitions.

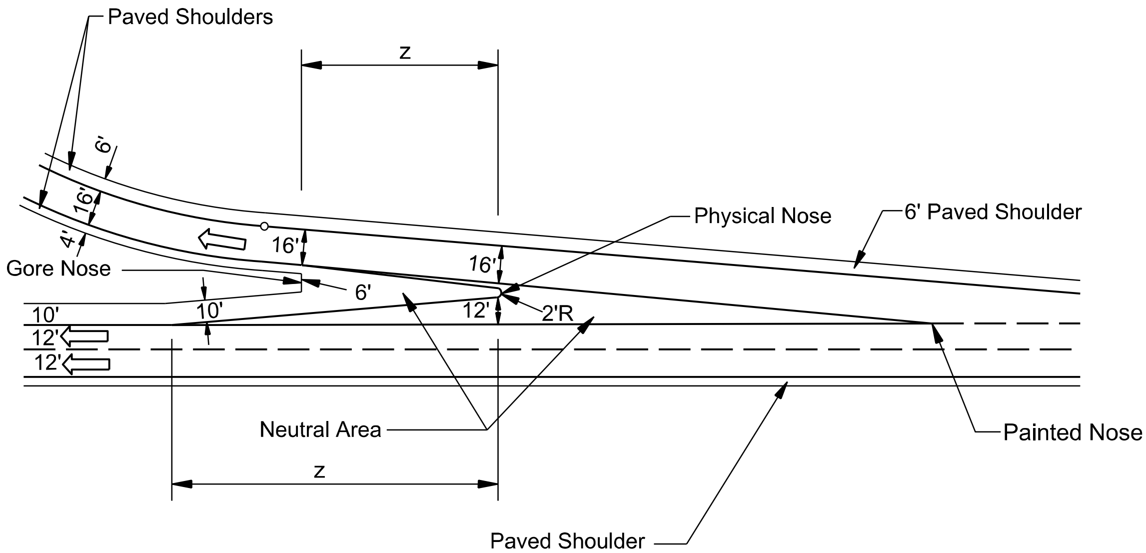
10.4.1.7 Shoulders

The wider right shoulder of the mainline must be transitioned to the narrower shoulder of the ramp (i.e., 10 foot paved to 6 foot paved). The shoulder width should be transitioned as shown in Figures 10.4-A and 10.4-B.

10.4.1.8 Gore Area

The gore area is normally considered both the paved triangular area between the through lane and the exit ramp, plus the graded area that may extend a few hundred feet downstream beyond the gore nose. The following definitions will apply (see Figure 10.4-F):

1. Painted Nose. This is the point (without width) where the pavement striping on the left side of the ramp converges with the stripe on the right side of the mainline traveled way.
2. Physical Nose. This is the point where the ramp and mainline shoulders converge. As illustrated in Figure 10.4-F, the physical nose is rounded with a 2-foot radius.
3. Gore Nose. This is the point where the paved shoulder ends and the grassed area begins as the ramp and mainline diverge from one another, as illustrated in Figure 10.4-F.



GORE AREA CHARACTERISTICS
Figure 10.4-F

Consider following when designing the gore:

1. Obstacles. If practical, the area beyond the gore nose should desirably be free of all obstacles (except the ramp exit sign) for at least 100 feet beyond the gore nose. Any obstacles within approximately 300 feet of the gore nose must be made breakaway or shielded by a barrier or impact attenuator.

2. Transitions. Figure 10.4-G provides the minimum taper rates (z) that should be used to transition the physical nose offset to the normal traveled way width.

3. Side Slopes. The graded area beyond the gore nose should be as flat as practical. If the elevation between the exit ramp or loop and the mainline increases rapidly, this may not be practical. These areas will likely be non-traversable, and the gore design must shield the motorist from these areas. At some sites, the vertical divergence of the ramp and mainline will warrant protection for both roadways beyond the gore; see AASHTO *Roadside Design Guide*.

4. Cross Slopes. The paved triangular gore or neutral area between the through lane and exit ramp should be safely traversable. The cross slope is the same as that of the mainline (typically 2.00 percent) from the painted nose up to the physical nose. Beyond this point, the gore area is depressed with cross slopes of 2 to 4 percent. See Section 10.4.1.6 for criteria on breaks in cross slopes within the gore area.

Design Speed of Approach Highway (mph)	Taper Rate (z) for Offset Nose
40	20:1
45	22.5:1
50	25:1
55	27.5:1
60	30:1
65	32.5:1
70	35:1
75	37.5:1

MINIMUM TAPER RATES BEYOND THE OFFSET PHYSICAL NOSE
Figure 10.4-G

10.4.2 Entrance Ramps

10.4.2.1 Types

There are two basic types of entrance freeway/ramp junctions — taper design and parallel design; see Figures 10.4-H and 10.4-I. For most entrance ramps, the parallel design is preferred considering the following:

1. Level of Service. Where the level of service for the freeway/ramp merge approaches capacity, a parallel design can be lengthened to allow the driver more time and distance to merge into the through traffic.

2. Acceleration Length. Additional acceleration length can be more easily provided by the parallel design than the taper design.

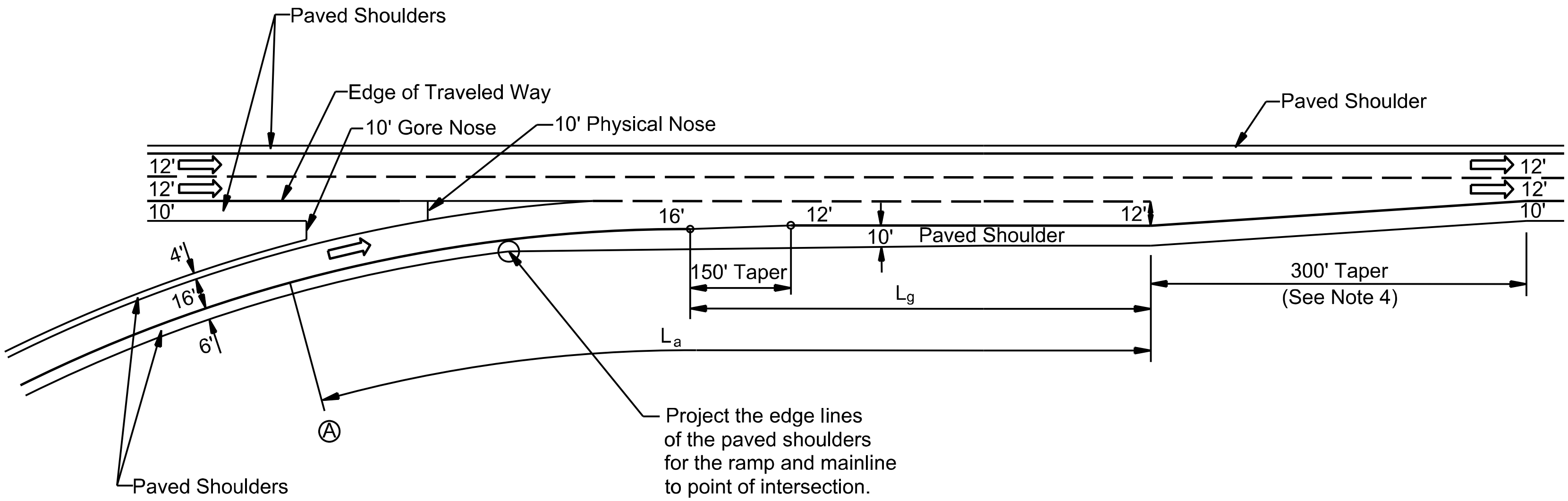
3. Sight Distance. Where there is insufficient sight distance available for the driver to merge into the mainline (e.g., where there are sharp curves on the mainline), the parallel entrance ramp allows a driver to use the side-view and rear-view mirrors to more effectively locate gaps in the mainline traffic.

10.4.2.2 Taper Rates

The following taper rates apply to the entrance design:

1. Parallel Design. For parallel-lane entrance ramps, the taper applies to the merge point at the end of the parallel portion of the ramp. The minimum distance is 300 feet as illustrated in Figure 10.4-H.

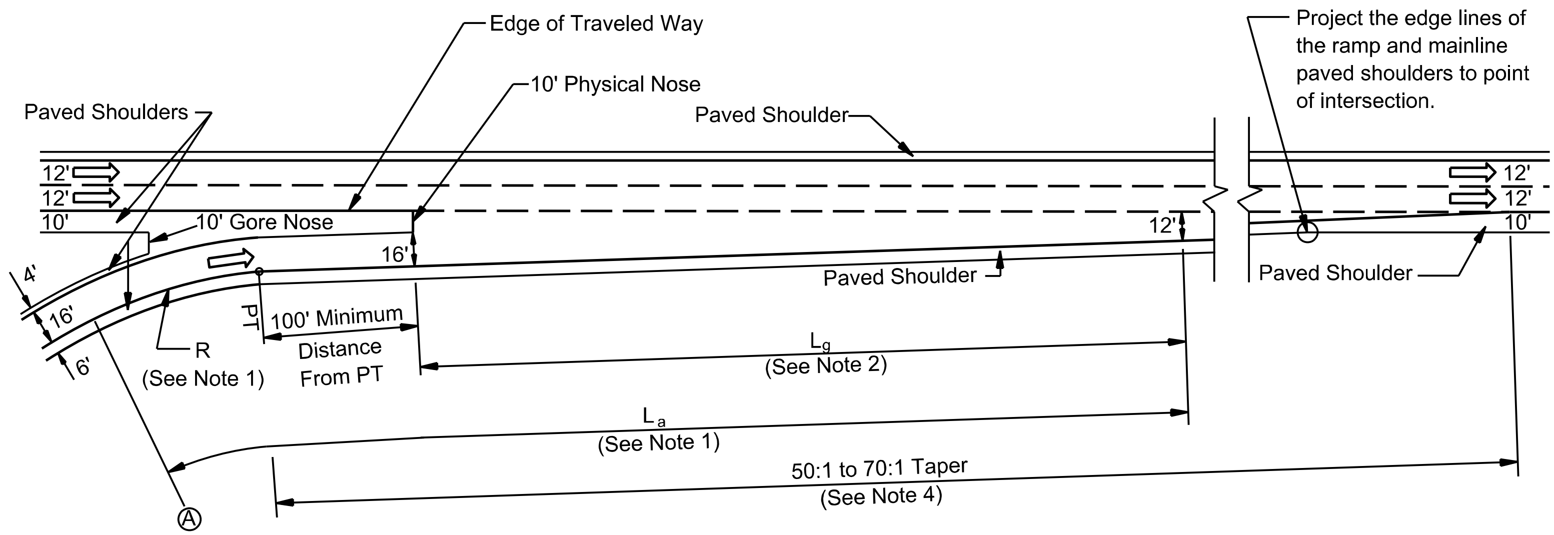
2. Taper Design. This rate applies to the rate at which the ramp connects with the mainline. The rate should be between 50:1 (minimum) and 70:1 (desirable) for merges onto a major highway and 25:1 for merges onto a crossroad. See Figure 10.4-I.



PARALLEL-LANE ENTRANCE RAMP
Figure 10.4-H

Notes:

1. *L_a is the required acceleration length. Point Ⓐ controls the safe speed on the ramp. L_a should not start on the curvature of the ramp unless the ramp radius is ≥ 1000 feet; see Section 10.4.2.3.*
2. *L_g is the required gap acceptance length. L_g should be a minimum of 300 feet.*
3. *Use the greater distance of L_a or L_g for determining the ramp entrance length.*
4. *The taper rate should be 50:1 to 70:1 if L_g is 2500 feet or greater; see Section 17.5.1.*



TAPER ENTRANCE RAMP
Figure 10.4-I

Notes:

1. *L_a is the required acceleration length. Point (A) controls the safe speed on the ramp. L_a should not start on the curvature of the ramp unless the ramp radius is ≥ 1000 feet; see Section 10.4.2.3.*
2. *L_g is the required gap acceptance length. L_g should be a minimum of 300 feet to 500 feet from the 10-foot nose width.*
3. *Use the greater distance of L_a or L_g for determining the ramp entrance length.*
4. *The transition taper rate of 50:1 to 70:1 is provided from the PT to the end of the taper.*

10.4.2.3 Acceleration

Driver comfort, traffic operations and safety will be improved if sufficient distance is available for acceleration. The length for acceleration will primarily depend upon the design speed of the last controlling horizontal curve on the entrance ramp and the design speed of the mainline. When determining the acceleration length, the designer should consider the following:

1. Passenger Cars. Figure 10.4-J provides the minimum lengths of acceleration for passenger cars. The acceleration distance is measured from the PT of the last controlling horizontal curve to the point at which the acceleration lane becomes less than 12 feet in width; see Figures 10.4-H and 10.4-I. Also, see Item 3 to determine how the horizontal curve interrelates with determining the acceleration distance. Where upgrades exceed 3 percent over the acceleration distance, adjust the acceleration length according to the values presented in Figure 10.4-K.

   The acceleration lengths provide sufficient distance for the acceleration of passenger cars. Where the mainline and ramp will carry traffic volumes approaching the design capacity of the merging area, the available acceleration distance should be at least 1200 feet, exclusive of the taper, to provide additional merging opportunities. This distance is measured from the PT of the ramp entrance curve.

2. Trucks. Where there are a significant number of trucks to govern the design of the ramp, consider providing the truck acceleration distances shown in Figure 10.4-L. Typical areas where trucks might govern the ramp design include weigh stations, rest areas, truck stops and transport staging terminals. At other freeway/ramp entrances, consider truck acceleration distances where there is substantial truck traffic and where:

   - there is a significant crash history involving trucks that can be attributed to an inadequate acceleration length, and/or

   - there is an undesirable amount of vehicular delay at the junction attributable to an inadequate acceleration length.

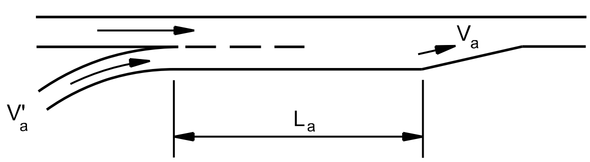
   Where upgrades exceed 3 percent, the truck acceleration distances may be adjusted for grades. Figure 6.4-B provides the performance criteria for trucks on descending and ascending grades. Before providing any additional acceleration length, the designer must consider the impacts of the added length (e.g., additional construction costs, wider structures, right-of-way impacts).

3. Horizontal Curves. In many cases, the speed of a vehicle entering the mainline from the ramp will be dictated by a horizontal curve immediately before the freeway/ramp junction. Determine the design speed of this horizontal curve using the criteria in Section 5.3. Use this speed to read into Figure 10.4-J or Figure 10.4-L to determine the necessary acceleration length.

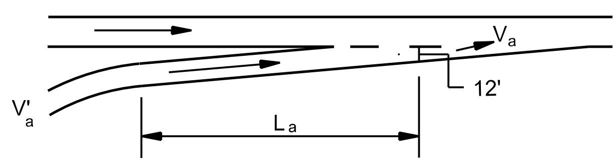
   Two exceptions to the above exist — entrance ramps at diamond interchanges and short entrance ramps. In these cases, determine the acceleration distance by that distance needed to accelerate from zero (at the beginning of the ramp) to the mainline design speed. The designer should check to determine if this distance governs.

In all cases, the curve preceding the freeway/ramp entrance should have a radius of 1000 feet or greater.

Design Speed of Highway (mph)	Speed Reached at End of Full Lane Width (mph) (V_a)	L_a = Length of Acceleration (ft)								
		For Entrance Curve Design Speed (mph)								
		Stop	15	20	25	30	35	40	45	50
		For Average Running Speed (mph) (V'_a)								
		0	14	18	22	26	30	36	40	44
30	23	180	140	—	—	—	—	—	—	—
35	27	280	220	160	—	—	—	—	—	—
40	31	360	300	270	210	120	—	—	—	—
45	35	560	490	440	380	280	160	—	—	—
50	39	720	660	610	550	450	350	130	—	—
55	43	960	900	810	780	670	550	320	150	—
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780



Parallel Type



Taper Type

Notes:

1. *The acceleration lengths are calculated from the distance needed for a passenger car to accelerate from the average running speed of the entrance curve to reach a speed (V_a) of approximately 5 miles per hour below the average running speed on the mainline.*
2. *These values are for grades less than 3 percent. See Figure 10.4-K for adjustments for steeper upgrades.*
3. *Select the actual design speed of the mainline and ramp when using this figure.*

LENGTHS FOR ACCELERATION
(Passenger Cars)
Figure 10.4-J

Design Speed of Highway (mph)	Ratio of Acceleration Length on Grade (G) to Length on Level For Entrance Curve Design Speed (mph)				
	20	30	40	50	All Speeds
	3% ≤ G ≤ 4% Upgrade				3% ≤ G ≤ 4% Downgrade
40	1.3	1.3	—	—	0.7
45	1.3	1.35	—	—	0.675
50	1.3	1.4	1.4	—	0.65
55	1.35	1.45	1.45	—	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
	4% < G ≤ 6% Upgrade				4% < G ≤ 6% Downgrade
40	1.5	1.5	—	—	0.6
45	1.5	1.6	—	—	0.575
50	1.5	1.7	1.9	—	0.55
55	1.6	1.8	2.05	—	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5

Notes:

1. *No adjustment is needed on grades less than 3 percent.*
2. *The grade in the table is the average grade measured over the distance for which the acceleration length applies. See Figures 10.4-H and 10.4-I.*

GRADE ADJUSTMENTS FOR ACCELERATION
(Passenger Cars)
Figure 10.4-K

* * * * * * * * * *

Example 10.4-2

Given: Highway Design Speed = 70 miles per hour
Entrance Ramp Curve Design Speed = 40 miles per hour
Average Grade = 5 percent upgrade

Problem: Determine length of acceleration required.

Solution: Figure 10.4-J yields an acceleration length of 1000 feet on the level. According to Figure 10.4-K, this should be increased by a factor of 2.6.

Therefore: L = (1000)(2.6)
L = 2600 feet

Provide a 2600-foot acceleration length from the PT of the entrance ramp curve to the beginning of the taper.

* * * * * * * * * *

Highway Design Speed (mph) (V)	Speed Reached (mph) (V_a)	L_a = Acceleration Length (ft)						
		For Entrance Curve Design Speed (mph)						
		Stop	15	20	25	30	35	40
		For Average Running Speed (mph) (V'_a)						
		0	14	18	22	26	30	36
55*	38	700	600	575	550	500	425	200
60	42	1300	1200	1175	1150	1100	1025	800
65	45	2100	2000	1975	1950	1900	1825	1600
70	48	2800	2700	2675	2650	2600	2525	2300

**For 55 miles per hour, the minimum lengths for passenger cars in Figure 10.4-J will apply.*

Notes:

1. *The acceleration lengths are calculated from the distance needed for a 200-pound per horsepower truck to accelerate from the average running speed of the entrance curve to reach a speed (V_a) that is 10 miles per hour below the average running speed on the mainline.*

2. *The taper entrance ramp is generally not applicable where trucks govern the design.*

LENGTHS FOR ACCELERATION
(200-Pound per Horsepower Truck)
Figure 10.4-L

10.4.2.4 Sight Distance

Provide drivers on the mainline approaching an entrance terminal sufficient distance to see the merging traffic so that they can adjust their speed or change lanes to allow the merging traffic to enter the freeway. Likewise, drivers on the entrance ramp need to see a sufficient distance upstream from the entrance to locate gaps in the traffic stream for merging. Therefore, provide decision sight distance according to the criteria in Section 4.3.

10.4.2.5 Superelevation

The entrance ramp superelevation should be gradually transitioned to meet the normal cross slope of the mainline. Apply the principles and criteria for superelevation as discussed in Section 5.3 to the entrance design. Section 10.4.1.5 provides the superelevation criteria for exit freeway/ramp junctions, which are also applicable to entrance freeway/ramp junctions. This includes e_{max}, design superelevation rate (e_d), transition lengths and point of revolution.

10.4.2.6 Cross Slope Rollover

The cross slope rollover is the algebraic difference between the slope of the through lane and the slope of the entrance ramp, where these two are adjacent to each other. The maximum algebraic difference is 5 percent beyond the physical nose. Between the gore nose and physical nose, the maximum cross slope rollover is 7 percent. See Section 10.4.3 for gore area definitions.

10.4.2.7 Shoulder Transitions

At entrance terminals, the right shoulder must be transitioned from the narrower ramp shoulder to the wider freeway shoulder (i.e., 6 feet to 10 feet). Figures 10.4-H and 10.4-I illustrate the typical shoulder transition. Note: Include taper rates for total shoulder width (paved and earth).

10.4.3 Gore Area

The following presents the nose dimensions for entrance gores:

1. Painted Nose. The painted nose dimension is considered to be 0.0 feet (i.e., the point where the two paint lines meet).

2. Physical Nose. The physical nose has a dimensional width of 10 feet, which is where the inside ramp edge meets the outside edge of the 10-foot paved freeway shoulder.

3. Gore Nose. The gore nose is where the outside edges of the ramp and mainline paved shoulders are 10 feet apart.

10.4.4 Multilane Terminals

Multilane terminals may be required when the capacity of the ramp is too great for single-lane operation. They may also be used to improve traffic operations (e.g., weaving) at the junction. The following lists several elements the designer should consider when a multilane terminal is required:

1. Lane Balance. Maintain lane balance at the freeway/ramp junction; see Section 10.3.4.

2. Entrances. For multilane entrance ramps, desirably use the parallel-lane design; however, a taper design may be considered.

3. Exits. For a multilane exit ramp, the additional lane should be at least 1500 feet prior to the terminal. The total length from the beginning of the first taper to the gore nose will range from 2500 feet for turning volumes of 1500 vehicles per hour or less up to 3500 feet for turning volumes of 3000 vehicles per hour.

4. Signing. Because of the complicated signing that may be required in advance of the exit, coordinate the geometric layout of multilane exits with the traffic designer.

10.4.5 Major Fork/Branch Connections

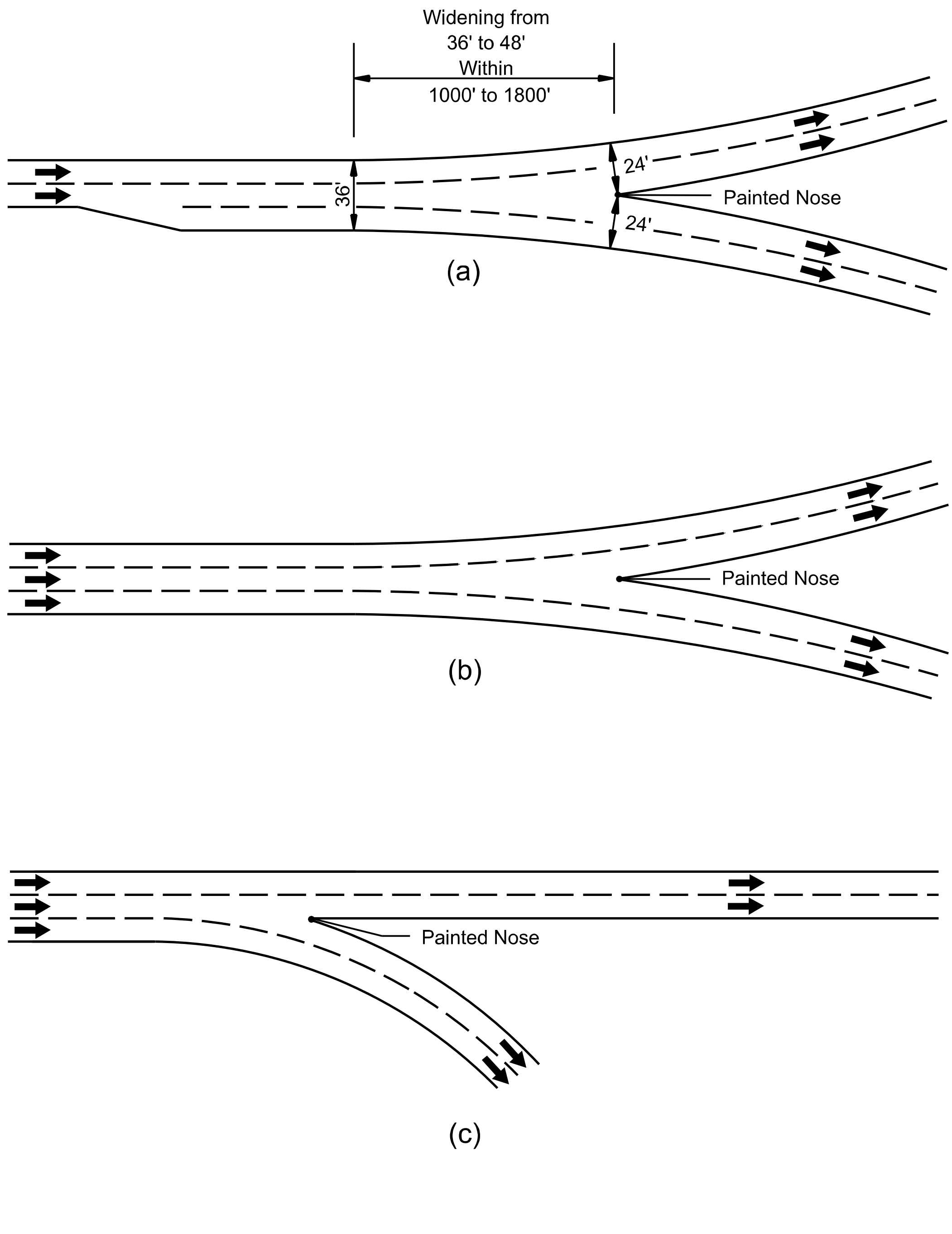
Figures 10.4-M and 10.4-N illustrate typical design details for a major fork or branch connection. The following presents a few geometric issues that the designer should consider when designing major divisions:

1. Lane Balance. Maintain the principle of lane balance; see Section 10.3.4.

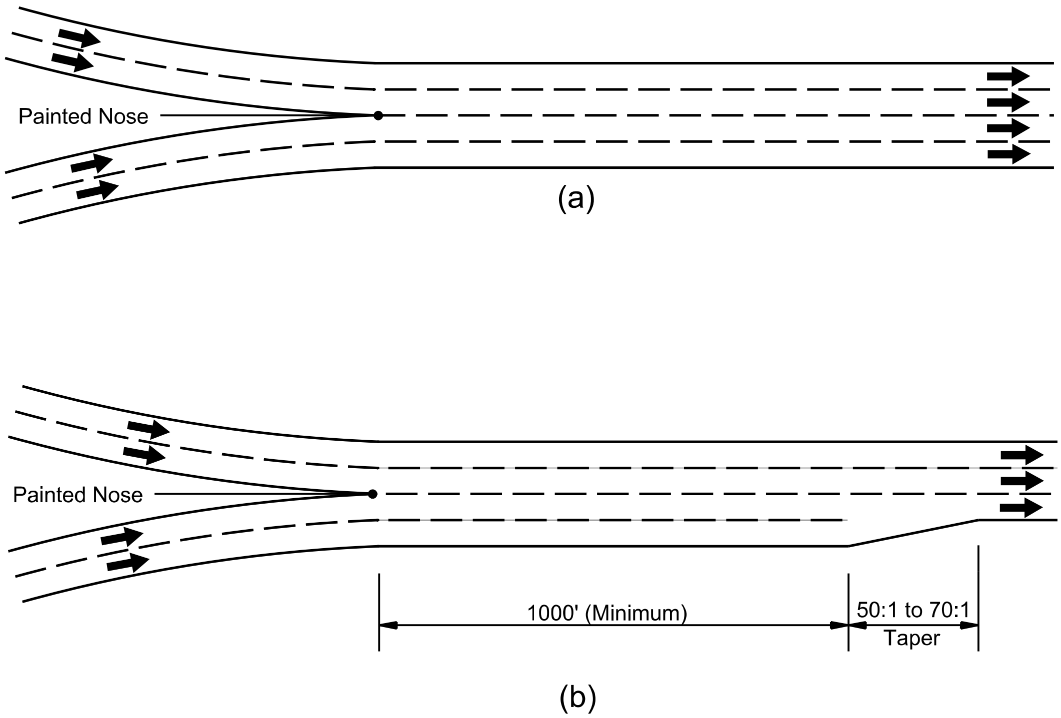
2. Divergence Point. Where the alignments of both roadways are on horizontal curves at a major fork, place the painted nose of the gore in direct alignment with the centerline of one of the interior lanes. This provides a driver in the center lane the option of going in either direction. See Figure 10.4-M(a) and (b). Where one of the roadways is on a tangent at a major fork, the gore design should be the same as a freeway/ramp multilane exit. See Figure 10.4-M(c).

3. Nose Width. At the painted nose of a major fork, the lane should be at least 24 feet wide, but preferably not more than 28 feet. The widening from 12 feet to 24 feet should occur within a distance of 1000 feet to 1800 feet. See Figure 10.4-M(a).

4. Branch Connection. When merging, provide a full lane width for at least 1000 feet beyond the painted nose. See Figure 10.4-N(b).



MAJOR FORKS
Figure 10.4-M



BRANCH CONNECTIONS
Figure 10.4-N

10.5 RAMP DESIGN

For design purposes, the ramp proper is assumed to begin at the gore nose for exit ramps and end at the gore nose for entrance ramps.

10.5.1 Ramp Types

The components of a ramp include the freeway/ramp junction, the ramp proper, and a free-flow or controlled ramp terminal at the crossroad. Although ramps have varying shapes, each can be classified into one or more of the types as illustrated in Figure 10.5-A and discussed in the following sections.

10.5.1.1 Loop Ramps

There are two types of loop ramps:

1. Free-Flow. The free-flow loop, Figure 10.5-A(a), consists of compounded circular arcs that turn through approximately 270 degrees. The free-flow loop is a standard component of the cloverleaf interchange, four-quadrant partial cloverleaf interchange and trumpet interchange. Free-flow loops are designed so that either the central arc is a sharper radius than that of the initial and final arcs or the central arc is intermediate between the two. Motorists decelerate from the speed of the through highway over the initial portion of the ramp and accelerate uniformly over the final portion of the ramp.

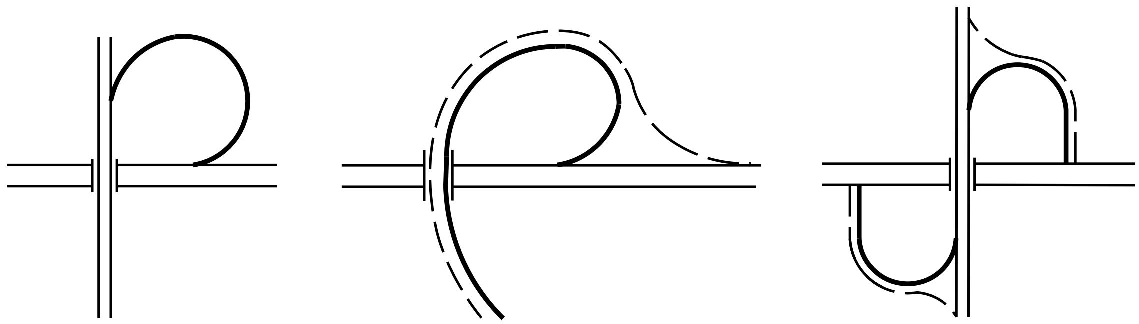
2. Controlled Terminal. Controlled terminal loops, Figure 10.5-A(b), are a component of the partial cloverleaf interchange. Controlled terminals are provided at the intersections with the crossroad and permit both right- and left-turning movements. Wherever practical, the angle of intersection should be 90 degrees.

10.5.1.2 Diagonal Ramps

Diagonal ramps, Figure 10.5-A(c), are components of the diamond interchange. Controlled terminals are provided on the crossroad. The angle of intersection with the crossroad varies between 60 degrees and 90 degrees.

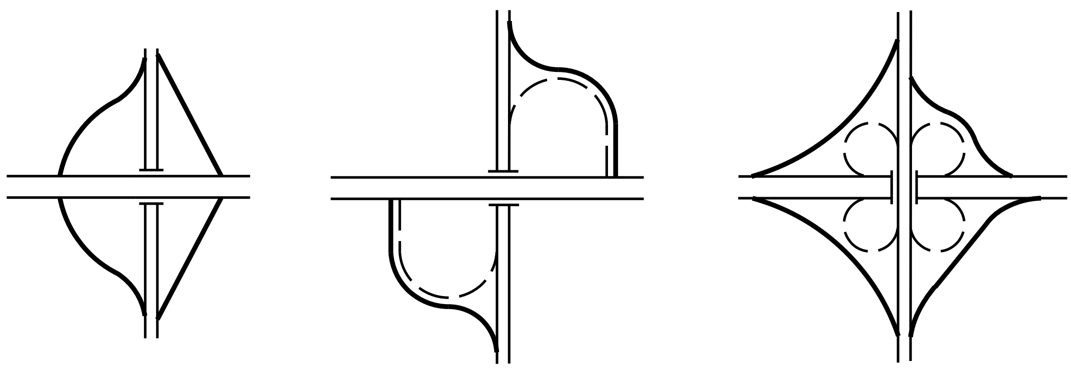
10.5.1.3 Outer-Connector Ramps

Outer-connector ramps are in the same quadrant and to the outside of loop ramps; see Figure 10.5-A(d). They may have free-flow operation (e.g., at cloverleaf or trumpet interchanges) or have controlled operations (e.g., at partial cloverleaf interchanges).



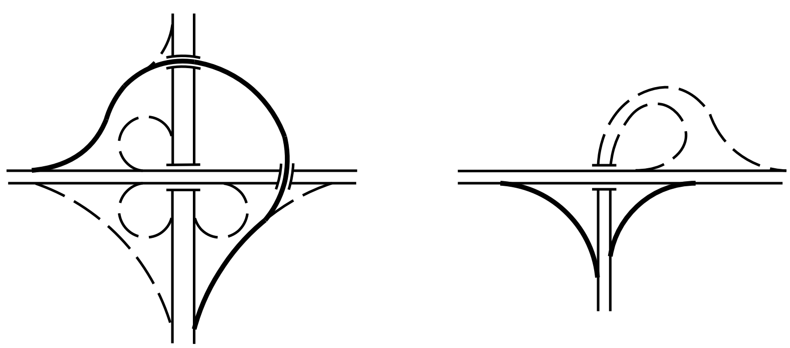
(a) Free-Flow Loop Ramps

(b) Controlled-Loop Ramps



(c) Diagonal Ramps

(d) Outer-Connector Ramps



(e) Semi-Direct Connection

(f) Direct Connection

Note: The heavier solid line indicates the ramp type being addressed.

RAMP TYPES
Figure 10.5-A

10.5.1.4 Semi-Direct Connections

Semi-direct connections are indirect in alignment, yet more direct than a loop ramp. These ramps are illustrated in Figure 10.5-A(e). Motorists making a left turn normally exit to the right and initially turn to the right, reversing direction before entering the intersecting highway. The outer ramp of the trumpet interchange is also a semi-direct connection.

10.5.1.5 Direct Connections

Direct connections do not deviate greatly from the intended direction of travel. These are illustrated in Figure 10.5-A(f) as an element of a trumpet interchange. They are also used to accommodate single-lane and right-turning traffic on four-quadrant partial cloverleafs and directional interchanges.

10.5.2 Design Speed

Figure 10.5-B provides the recommended ranges of ramp design speeds based on the design speed of the mainline. In addition, consider the following when selecting the ramp design speed:

1. Loop Ramps. Design speeds in the middle and upper ranges are generally not attainable for loop ramps. The following apply to loop ramps:

    a. For loop ramps on collector-distributor roadways or in restricted urban conditions, the minimum design speed for loops should be 25 miles per hour.

    b. Where the truck ADT is greater than 15 percent, use a minimum design speed of 30 miles per hour for the initial curve after the exit curve.

    c. For rural loop ramps, use a minimum design speed of 30 miles per hour.

    d. Use a design speed of 40 miles per hour for cloverleaf interchange loop ramps between freeways used in conjunction with C-D roads.

	Mainline Design Speed				
	55 mph	**60 mph**	**65 mph**	**70 mph**	**75 mph**
	Ramp Design Speed (mph)				
Upper Range	45-50	50	55	60	65
Middle Range	40	45	45	50	55
Lower Range	25-30	30	30	35	40

RAMP DESIGN SPEEDS
Figure 10.5-B

2. <u>Outer Connector Ramps</u>. The design speed for the outer connector ramp of a rural cloverleaf interchange should be 50 miles per hour. Where a wrap-around type ramp is used, use a minimum design speed of 45 miles per hour for the center curve.

3. <u>Semi-Direct Connections</u>. Use design speeds in the middle to upper ranges for semi-directional ramps. Do not use a design speed less than 30 miles per hour.

4. <u>Direct Connections</u>. These include both diagonal ramps at a diamond interchange and ramps at a directional interchange. Use a design speed in the middle to upper ranges. Do not use a design speed less than 40 miles per hour.

5. <u>Controlled Terminals</u>. If a ramp is terminated at an intersection with a stop or signal control, the design speeds in Figure 10.5-B are not applicable to the portion of the ramp near the intersection. The design speed on the ramp near the crossroad intersection can be a minimum of 25 miles per hour.

6. <u>Variable Speeds</u>. The ramp design speed may vary based on the two design speeds of the intersecting roadways (i.e., mainline design speed and crossroad design speed). The selected design speed should be consistent with the connecting facilities. When using multiple ramp design speeds, the maximum speed differential between controlling design elements (e.g., horizontal curves, vertical curves) should not be greater than 10 to 15 miles per hour. The designer must ensure that there is sufficient deceleration distance available between design elements with different design speeds (e.g., two horizontal curves). See Section 10.5.5.

Figure 10.5-C presents geometric design criteria for interchange ramps based on the selected design speed (e.g., sight distance, horizontal and vertical alignment).

10.5.3 <u>Sight Distance</u>

The designer should ensure that stopping sight distance is continuously provided along the ramp. Because ramps are composed of curves of various radii and design speeds, sight distance requirements may vary over the length of the ramp. The designer should provide decision sight distance at locations with complex maneuvers. Figure 10.5-C provides a summary of the geometric criteria for ramps, including stopping sight distance.

10.5.4 <u>Cross Section Elements</u>

Figure 10.5-D presents the typical cross sections for tangent and loop ramps. The following also applies to the ramp cross section:

1. <u>Width</u>. The total paved ramp width will be the sum of the ramp traveled way, the left shoulder and the right shoulder. For most ramps, the typical ramp traveled way is 16 feet. For locations with significant numbers of trucks and tight radii, consider widening the ramp shoulders. If the facility has unpaved shoulders, review the ramp shoulder criteria from the AASHTO *A Policy on Geometric Design of Highways and Streets* to determine the applicable ramp width. Assume the Case II and “C” design traffic conditions.

Geometric Requirements									
RAMP DESIGN SPEED (mph)	65	60	55	50	45	40	35	30	25
STOPPING SIGHT DISTANCE (ft)	645	570	495	425	360	305	250	200	155
HORIZONTAL ALIGNMENT									
Minimum Radius (ft) e_{max} = 8%	1480	1200	960	758	587	444	314	214	134
Minimum Length of Arc (ft)	See Figure 10.5-E								
VERTICAL ALIGNMENT									
Maximum Grades	3-5%	3-5%	3-5%	3-5%	3-5%	4-6%	4-6%	5-7%	5-7%
Crest Vertical Curves (K-values)*	193	151	114	84	61	44	29	19	12
Sag Vertical Curves (K-values)*	157	136	115	96	79	64	49	37	26

**K-values are based on stopping sight distance on level grades.*

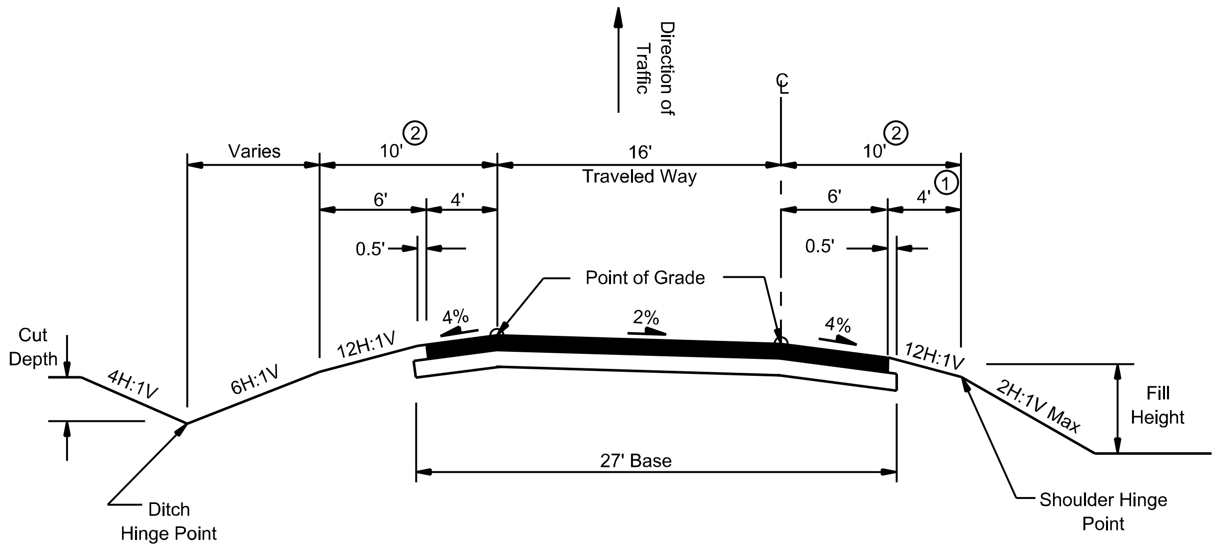
ALIGNMENT CRITERIA FOR INTERCHANGE RAMPS
Figure 10.5-C

The typical right-paved shoulder is 6 feet, and the typical left-paved shoulder is 4 feet in the direction of travel. The left shoulder-traveled way-right shoulder arrangement is illustrated in the ramp cross sections in Figure 10.5-D. For multilane directional ramps, the cross sectional width is the same as the mainline design (e.g., 24-foot traveled way width plus shoulders); see Chapter 17 “Freeways.”

2. Cross Slope. For tangent sections, the ramp traveled way is sloped unidirectional at 2.00 percent towards the right shoulder. Shoulder cross slopes on tangent are typically 4.00 percent. The left shoulder is typically sloped away from the traveled way.

3. Curbs. Only use curbs on urban interchange ramps and only where necessary. If curb and gutter is required for drainage, use sloping curb and place it on the outside edge of the full-width paved shoulders. See Section 7.2.8 for information on the use of curbs.

4. Bridges and Underpasses. Carry the full width of the ramp (shoulders and travel lanes), across the bridge. See Chapters 7 “Cross Section Elements” and 17 “Freeways” when determining the clear ramp width for an underpass.



Notes:

① *Add 3.5 feet where guardrail is used.*

② See Section 5.3 for maximum shoulder break.

TYPICAL RAMP CROSS SECTIONS
Figure 10.5-D

5. Side Slopes/Ditches. For the ramp proper, side slopes and ditches should meet the same criteria as for the highway mainline. Chapters 7 “Cross Section Elements” and 17 “Freeways” provide the applicable design information for side slopes and ditches.

6. Roadside Safety. See the AASHTO *Roadside Design Guide* for clear zone criteria and barrier warrants, selection and layout.

7. Right of Way. The right of way adjacent to the ramp is fully access controlled and the right of way is typically fenced. See Chapter 12 “Right of Way.”

10.5.5 Horizontal Alignment

The following will apply to the horizontal alignment of ramps:

1. Minimum Curve Radii. Figure 10.5-C provides the minimum curve radii based on ramp design speed and e_{max}.

2. Superelevation Rates. See Section 5.3 for superelevation rates based on design speed, design superelevation and curve radius.

3. Curve Type. On all ramps, except loop ramps, only use simple curves unless field constraints (e.g., to avoid an obstruction) dictate the use of compound curvature. On loop ramps, the designer should typically use compound curves with the interior curve(s) of sharper radii than the exterior curves. For exits with loops, the radii of the flatter arc

compared to the radii of the sharper arc should not exceed a ratio of 2:1 to prevent abruptness in operation and appearance. Where compound arcs of decreasing radii are used, the arcs should have sufficient length to enable motorists to decelerate at a reasonable rate over the range of design speeds. See Figure 10.5-E.

Comparable radii and length controls may be used on entrance loop ramps with compound arcs of increasing radii. However, for entrance ramps, the 2:1 ratio of compound curves and the lengths in Figure 10.5-E is not as critical because the vehicle is accelerating into a curve with a larger radius or into a tangent section.

Radius (ft)	**100**	**150**	**200**	**250**	**300**	**400**	**500 or more**
Minimum (ft)	40	50	60	80	100	120	140
Desirable (ft)	60	70	90	120	140	180	200

Note: These lengths are applicable to ramp curves followed by a curve 1/2 its radius or preceded by a curve of double its radius.

ARC LENGTHS FOR COMPOUND CURVES
Figure 10.5-E

4. Trucks. Where there are a significant number of trucks on loop ramps, the designer should consider how the design may impact the rollover potential for large trucks. To reduce this potential, consider using flatter curve radii and/or a higher ramp design speed than the allowable minimums. Other critical factors include ensuring that ample deceleration lengths are available and, if judged necessary by the traffic designer, include special rollover warning signs for trucks.

5. Baseline/Centerline. The following will apply:

   a. Ramp. Typically, the outside edge (away from the interior of the interchange) of the ramp traveled way is used for horizontal and vertical control. This edge may be used for the point of grade/revolution or the designer may choose to use the inside edge.

   b. Loop. Typically, the inside edge (near the interior of the interchange) of the loop traveled way is used for horizontal and vertical control. This edge may be used for the point of grade/revolution or the designer may choose to use the outside edge.

6. Controlled Ramp Termini. Exit ramps may end at a controlled intersection — stop control or signal control. See Chapter 9 “Intersections.”

10.5.6 Vertical Alignment

10.5.6.1 Grades

Maximum grades for vertical alignment cannot be as definitively expressed as those for the highway mainline. General values of limiting gradient are shown Figure 10.5-C, but for any one ramp the selected profile is dependent upon a number of factors. These factors include:

1. The flatter the gradient on the ramp, the longer the ramp will be. At restricted sites (e.g., loops), it may be advantageous to provide a steeper grade to shorten the ramp length.

2. Use the steepest gradients for the center portion of the ramp. Freeway/ramp junctions and landing areas at intersections should be as flat as practical.

3. Short upgrades up to 5 percent do not unduly interfere with truck and bus operations. Consequently, for new construction it is desirable to limit the maximum gradient to 5 percent.

4. Downgrades on ramps should follow the same guidelines as upgrades. However, where there are sharp horizontal curves and significant truck and bus traffic, it is desirable to limit the downgrades to 3 to 4 percent.

5. The ramp grade within the freeway/ramp junction up to the physical nose should be approximately the same grade as that provided on the mainline.

10.5.6.2 Vertical Curvature

Design vertical curves on ramps to meet the stopping sight distance criteria based on the ramp design speed as presented in Section 6.5. Figure 10.5-C provides the K-values for both crest and sag vertical curves on level grades. The ramp profile often assumes the shape of the letter S with a sag vertical curve at one end and a crest vertical curve at the other. In addition, where a vertical curve extends onto the freeway/ramp junction, determine the length of curve using a design speed comparable to the mainline. See Section 6.5 for details on the design of vertical curves.

10.5.6.3 Cross Sections Between Adjacent Ramps

Where the alignment of a ramp is designed to be parallel to an adjacent ramp (e.g., cloverleaf, trumpet interchanges), first establish the profile of the loop ramp and then set the profile of the outer ramp to be approximately parallel to the inner-loop ramp profile. Accomplish this by calculating the left-edge elevations of the loop ramp and matching those elevations for the left-edge elevations of the outer ramp.

10.5.7 Ramp Merges

The designer needs to give special consideration to the design where two ramps merge to together (e.g., directional interchange ramps), which are designed on a case-by-case basis. These merges will require coordination with the traffic designer to conduct an operational analysis.

10.6 RAMP/CROSSROAD INTERSECTION

Chapter 9 “Intersections” presents the Department’s in-depth criteria on the design of at-grade intersections. This section presents additional information that is applicable to the intersection of an interchange ramp and the crossroad.

At diamond and partial cloverleaf interchanges, the ramp will terminate or begin with an at-grade intersection. In general, design the intersection as described in Chapter 9. This will involve a consideration of capacity and the physical geometric design elements (e.g., sight distance, angle of intersection, acceleration lanes, channelization and turning lanes). The designer should also consider the following in the design of the ramp/crossroad intersection:

1. Crossroad Width. The crossroad width will be based on the anticipated traffic volumes for the design year, the crossroad functional classification and the design criteria presented in Chapter 9 “Intersections.”

2. Sight Distance. Section 4.4 discusses the criteria for intersection sight distance (ISD). Ramp/crossroad intersections present unique ISD problems because of the nearby bridge structure at most interchanges. Give special consideration to the location of bridge piers, abutments, sidewalks, bridge rails, roadside barriers, etc.; these elements may present major sight obstructions. The bridge obstruction and the required ISD may result in the relocation of the ramp/crossroad intersection further from the structure. Also, crest vertical curves on the crossroad may need to be flattened to provide adequate sight distance in the vertical plane.

3. Capacity. In urban areas where traffic volumes are often high, inadequate capacity of the ramp/crossroad intersection can adversely affect the operation of the ramp/freeway junction. In a worst-case situation, a backup onto the freeway may impair the safety and operation of the mainline itself. Therefore, give special attention to providing sufficient capacity and storage for an at-grade intersection or a merge with the crossing road. This may require providing additional lanes at the intersection or on the ramp proper, or it could involve a specialized design to reduce queuing. The analysis must also consider the operational impacts of the traffic characteristics in either direction on the intersecting road.

4. Turn Lanes. Exclusive left- and/or right-turn lanes often will be required on the crossroad and in many cases on the ramp itself. Chapter 9 “Intersections” provides information on the design of turn lanes at intersections.

5. Signalization. Where queuing at one intersection is long enough to affect operations at another, the two intersections may require a larger separation or coordinated signal design.

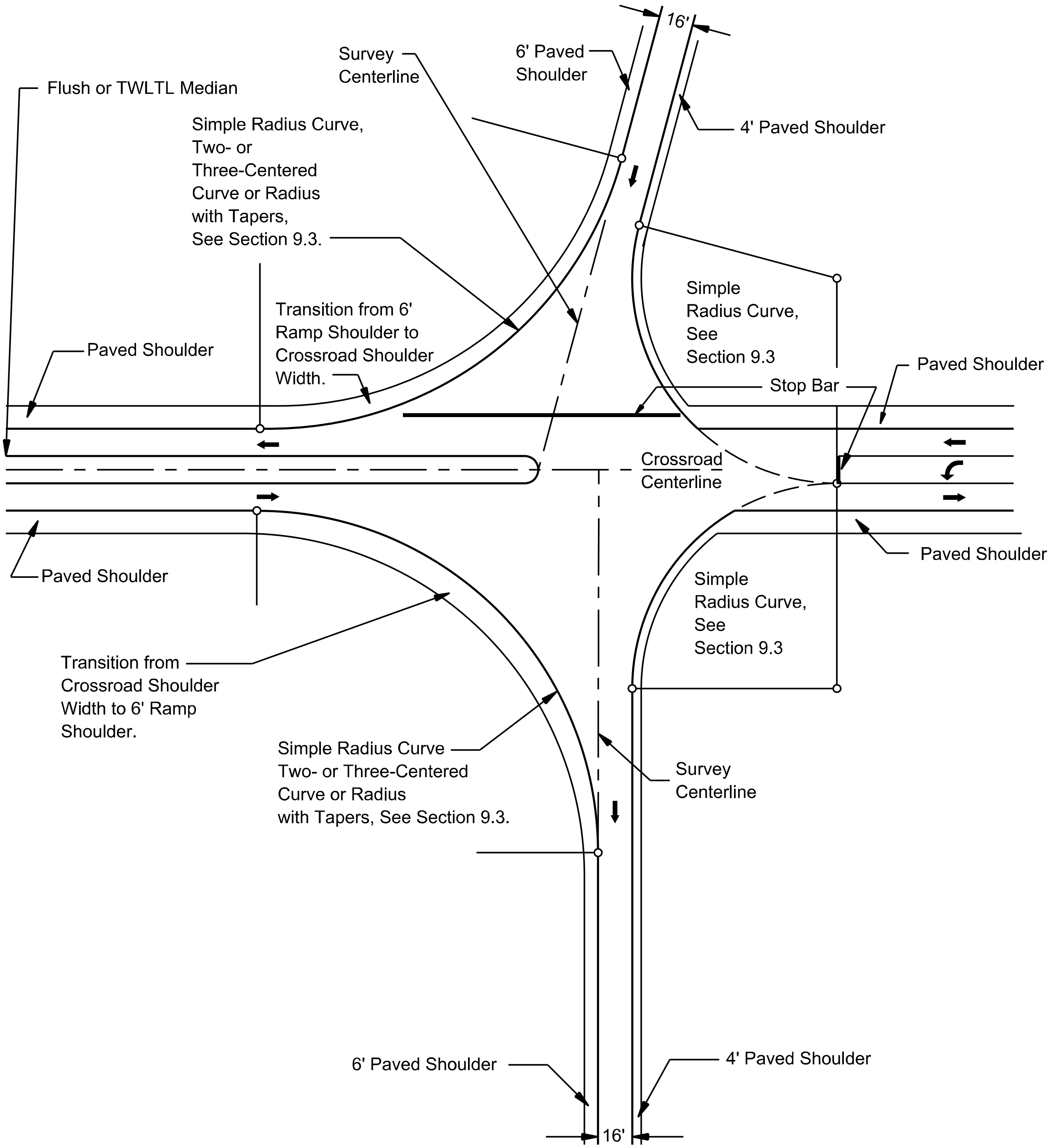
6. Design Vehicle. Design all radius returns and left-turn control radii for ramp/crossroad intersections using a WB-62 design vehicle; see Section 9.3. Use the WB-67 design vehicle for determining storage lengths (e.g., left-turn lanes), median widths, etc., for the ramp/crossroad intersection.

7. Typical Designs. Figures 10.6-A and 10.6-B illustrate typical ramp/crossroad intersections for a diamond interchange. Figure 10.6-A illustrates a three-lane crossroad

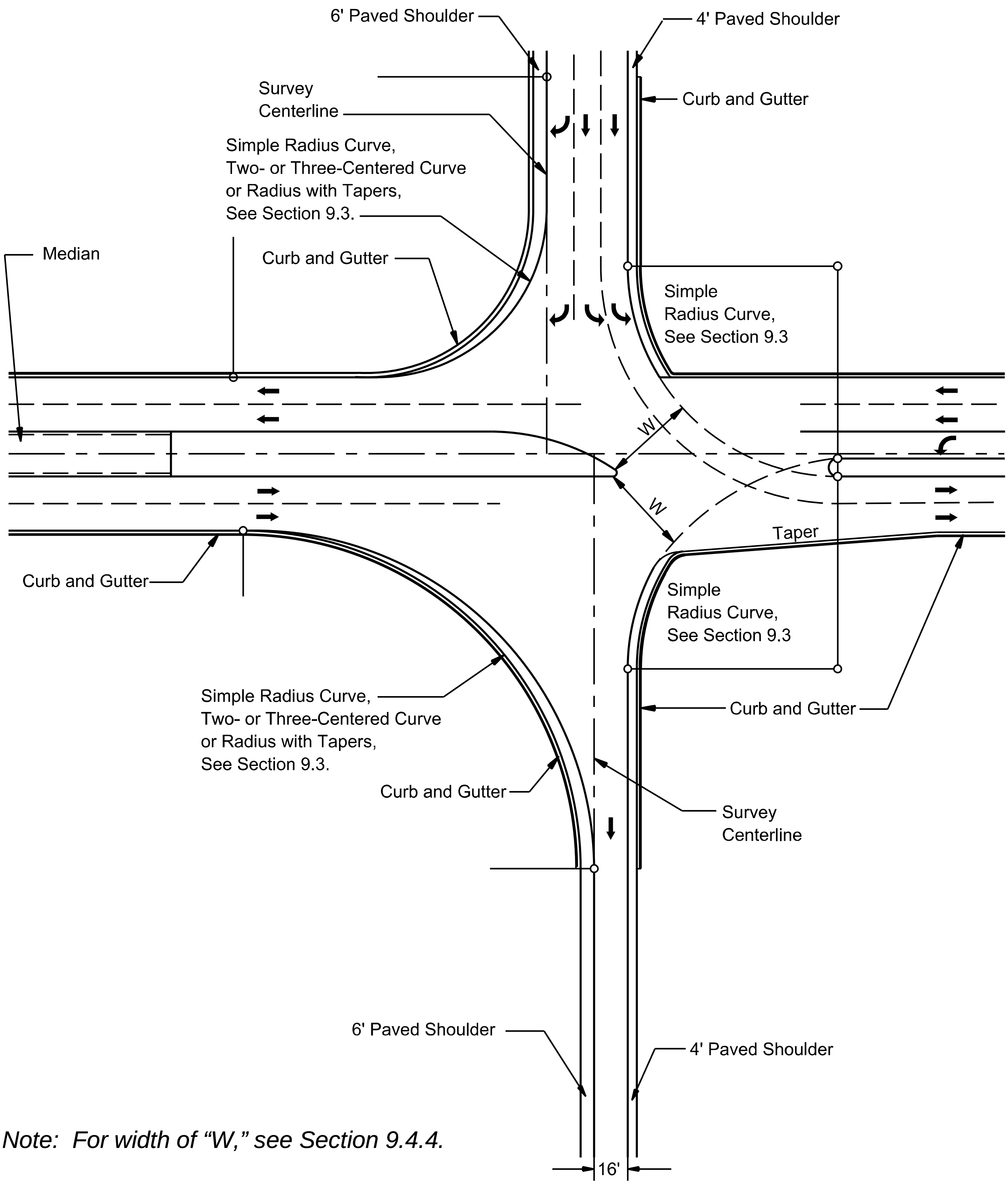
and Figure 10.6-B a four-lane multilane curb and gutter crossroad with a two-way, left-turn lane.

8. Wrong-Way Movements. Wrong-way movements often originate at the ramp/ crossroad intersection onto an exit ramp. To minimize their probability, design the intersection geometry to discourage this movement and provide appropriate access management techniques (e.g., raised medians).

9. Access Control. Section 3.8 presents detailed information on access control along the crossroad in the vicinity of ramp/crossroad intersections.



RAMP/CROSSROAD INTERSECTION—DIAMOND INTERCHANGE
(Three-Lane Crossroad)
Figure 10.6-A



Note: For width of "W," see Section 9.4.4.

RAMP/CROSSROAD INTERSECTION — DIAMOND INTERCHANGE
(Five-Lane TWLTL Crossroad — Signalized Intersections)
Figure 10.6-B

10.7 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. *A Policy on Design Standards Interstate System*, AASHTO, 2005.

3. *User and Non-User Benefit Analysis for Highways*, AASHTO, 2010.

4. *Federal Register*, "Additional Interchanges to the Interstate System," Vol. 74, No. 165, August 27, 2009.

5. *Interstate System Access Information Guide*, FHWA, August 2010.

6. *Freeway and Interchange: Geometric Design Handbook*, ITE, 2005.

7. NCHRP Report 345, *Single Point Urban Interchange, Design and Operations Analysis*, Transportation Research Board, 1991.

8. *Highway Capacity Manual 2010*, Transportation Research Board, 2010.

9. *Manual on Uniform Traffic Control Devices*, FHWA, ATSSA, AASHTO and ITE, 2009.

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Chapter 11

SPECIAL DESIGN ELEMENTS

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 11
SPECIAL DESIGN ELEMENTS

The designer must address numerous design elements that are not directly related to the geometric design of the roadway. This chapter provides a discussion on several of these design elements including accessibility requirements, retaining walls, landscaping, noise control, rest areas, park and ride facilities, managed lanes, bus stops/turn outs, mailboxes, cul-de-sacs and bicycle accommodations.

11.1 ACCESSIBILITY FOR INDIVIDUALS WITH DISABILITIES

Many highway elements can affect the accessibility and mobility of individuals with disabilities. These include pedestrian access routes, parking lots, buildings at transportation facilities, overpasses and underpasses. A pedestrian access route is a continuous and unobstructed path of travel provided for pedestrians with disabilities within or coinciding with a pedestrian circulation path. The pedestrian access routes are typically contained within the sidewalk.

The Department accessibility criteria complies with the 1990 *Americans with Disabilities Act* (ADA) and is provided in the Department's publication "Americans with Disabilities Act Transition Plan," which can be found on the Department's internet site. The ADA criteria provided in the SCDOT Transition Plan and the *SCDOT Standard Drawings* is based on the United States Access Board's *Public Right-of-Way Accessibility Guidelines* (PROWAG). The SCDOT Transition Plan sets forth the steps necessary to complete physical and other modifications of SCDOT facilities and programs for which it is responsible in order to achieve ADA required accessibility. The Plan includes the following:

- identification of certain physical obstacles that limit accessibility,
- description and details of the methods that will be used to make the facilities accessible,
- schedule for taking the steps necessary to achieve compliance with the ADA, and
- identification of the person responsible for implementation of the Plan.

Designers are required to meet the criteria presented in the Department's Transition Plan. Where other agencies or local codes require criteria that exceed the ADA Guidelines, then the stricter criteria may be required. This will be determined on a case-by-case basis.

The following definitions are used with ADA Guidelines:

1. Accessible. Describes a facility in the public right-of-way that complies with PROWAG.
2. Alteration. A change to a facility in the public right-of-way that affects or could affect access, circulation or use.
3. Facility. All or any portion of buildings, structures, improvements, elements and pedestrian or vehicular routes located in a public right-of-way.

4. Pedestrian Access Route (PAR). A continuous and unobstructed walkway within a pedestrian circulation path that provides accessibility.

5. Pedestrian Circulation Path. A prepared exterior or interior way of passage provided for pedestrian travel.

6. PROWAG. PROWAG is recognized as the Department's technical guide for design of ADA compliant pedestrian facilities.

7. Public Right-of-Way. Public land or property, usually in interconnected corridors, that is acquired for or devoted to transportation services.

8. Walkway. The continuous portion of the pedestrian access route that is connected to street crossings by curb ramps or blended transitions.

11.2 EARTH RETAINING STRUCTURES

Where increasing traffic requires a new roadway or the addition of lanes, earth retaining structures (ERS) are often necessary (e.g., where existing or proposed slopes are unstable and flattening of the slope is not reasonable). See the SCDOT *Geotechnical Design Manual* for definitions of ERS reinforced soil slope and reinforced and unreinforced embankments. The Program Manager and the Plan Production Team are responsible for identifying the need for an ERS (e.g., difference in grade elevations, slope stabilization, limited right of way available, environmental concerns, temporary excavation support). For information regarding right of way issues, see Chapter 12 "Right of Way." For ERSs and design, see the SCDOT *Geotechnical Design Manual* and the SCDOT *Bridge Design Manual*. Roadway designers should coordinate the design of retaining systems with the Program Manager, structural designer and geotechnical designer.

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11.3 LANDSCAPING

The following controlling principles are based upon the conservation of natural resources; creating a facility that is compatible with its surroundings; minimizing future management efforts and expenditures; and producing a high quality, environmentally responsible finished product:

1. Environmental Impact. Where practical, avoid adverse or disruptive impacts to landscape and environmental features on or adjacent to the right of way. Where total avoidance of adverse or disruptive impacts is not practical, the designer should undertake all reasonable measures to reduce and minimize impacts to these features. If damage or disruption is unavoidable, undertake all reasonable measures to offset damages through mitigation in the project area or other designated areas. Note that the designer cannot recreate or restore natural systems, but can use native plant materials to represent some of the appearances and functions of the impacted feature.

2. Environmentally Sensitive Areas. Consider environmentally sensitive areas and those harboring endangered species to be a controlling factor in all designs.

3. Use of Indigenous Plants. Emphasize the use of plants native to and grown in South Carolina that are appropriate to the site, its planned use and its future management.

4. Site Compatibility. Design a specific landscape that is compatible with the site. This includes design elements that may affect a landscape plan for a specific site (e.g., conduit for electric, water requirements, pipe requirements, vegetation type).

5. Future Maintenance Considerations. Consider the future maintenance plans for the roadside area to be a controlling factor in the planning and design of that area.

6. Sustainable Roadside Environment. Strive to produce a sustainable roadside environment.

7. Visual Quality. Visual appearance and visual unity of the facilities are important components. Recognize that visual quality must be a component in almost all design development and that numerous factors influence the final appearance of the finished project. Durability and appearance are the two items most noticed and commented upon by the traveling public.

8. Plants to Avoid. For a list of plants to avoid, see the Department’s internet site.

The designer should apply landscape and environmentally based design principles to the full range of highway types, from multi-lane freeways to the rehabilitation and improvement of existing local arterials and rural collector roads.

The extent of the application of these principles will depend on the type of project, the environmental resources affected and the public entities involved. For additional guidance, reference the Department’s landscaping guidelines found on the Department’s internet site, AASHTO *Roadside Design Guide,* AASHTO *A Guide for Transportation Landscape and Environmental Design* and *SCDOT Access and Roadside Management Standards Manual*.

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11.4 NOISE CONTROL

The SCDOT *Traffic Noise Abatement Policy* provides SCDOT noise abatement requirements with respect to 23 CFR 772, “Procedures for Abatement of Highway Traffic Noise and Construction Noise.” SCDOT recognizes the adverse effects that highway traffic noise may have on the citizens of South Carolina and does what is practical to lessen these effects. During the project development process various noise abatement options are considered to abate noise impacts including alternative alignments or noise structures. The SCDOT Environmental Services Office is responsible for determining if noise abatement measures are required.

The designer should carefully consider the construction and placement of noise barriers so they will not increase the severity of crashes that may occur. Ensure noise barriers are located to allow for sign placement and to provide lateral offsets to obstructions outside the edge of the traveled way. However, such a setback may sometimes be impractical. In these situations, provide the largest practical width commensurate with cost-effectiveness considerations.

Stopping sight distance may be an issue with noise barriers. Check horizontal clearances along the inside of a horizontal curve to ensure adequate sight distances are available. Some designs use a concrete safety shape either as an integral part of the noise barrier or as a separate roadside barrier between the edge of the roadway and the noise barrier. On non-tangent alignments, a separate concrete barrier may obstruct sight distance even though the noise barrier does not. In these instances, it may be appropriate to install metal rather than concrete roadside barriers in order to retain adequate sight distance.

Give special consideration of noise barriers near gore areas. Barriers at these locations should begin or terminate, as the case may be, at least 200 feet from the theoretical nose.

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11.5 REST AREAS

The primary responsibility of the State to motorists using highway systems is safety, and rest areas are an important instrument for highway improvement. Crash reduction is the primary function. Greater highway safety is the major benefit in establishing rest areas, through safe off-road locations for motorists to rest, sleep, change drivers and check vehicle loads and/or minor mechanical problems. Additional benefits for motorists are relief from extended travel period time, increased comfort and convenience, and locations for public agencies to communicate with travelers.

Well-designed, well-maintained rest areas also create positive images for out-of-state motorists and enhance quality of life for the South Carolina's own residents. They provide opportunities for SCDOT and tourism groups to communicate with motorists in promoting State and local programs, and to provide road and weather information and such directional services as maps, routing suggestions, traffic incident warnings and road construction schedules.

Roadway designers are typically responsible for the design of exit and entrance terminals, internal roadways and parking areas, and work with other SCDOT Sections (e.g., Traffic, Planning) to design and rehabilitate rest areas. For the design of exit and entrance terminals and internal roadways; see Chapter 10 "Interchanges." For the design of parking areas, see Section 7.2.7. For additional guidance on rest areas, see the AASHTO *Guide for Development of Rest Areas on Major Arterials and Freeways*.

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11.6 PARK-AND-RIDE FACILITIES

There are various Transportation Systems Management (TSM) tools to help achieve increased efficiency of traffic on existing transportation systems. Typical examples of TSM tools are:

- traffic operation improvements at intersections,
- operational improvements on freeways, and
- improved signalization and street systems control (one-way street or reversible lanes).

Implementation of the above TSM tools typically involves greater use of public transit (primarily buses), carpools, vanpools or other ridesharing modes, which requires the construction of park and ride facilities. The designer should review the *SCDOT Access and Roadside Management Standards Manual* and the AASHTO *Guide for the Design of Park-and-Ride Facilities* for guidance on the implementation and design of park-and-ride facilities.

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11.7 MANAGED LANES

Managed lanes are defined as highway facilities or a set of lanes where operational strategies are proactively implemented and managed in response to changing conditions. The managed lane concept is typically a “freeway-within-a-freeway” where a set of lanes within the freeway cross section is separated from the general-purpose lanes. Examples of operating managed lane projects include high-occupancy vehicle (HOV) lanes, value priced lanes, high occupancy toll (HOT) lanes, or exclusive or special use lanes. Each of these concepts offers unique benefits; therefore, careful consideration must be given to project goals and objectives in choosing an appropriate lane management strategy or combination of strategies.

For guidance on the design and implementation of managed lanes, see the FHWA document *Managed Lanes: A Primer*, which can be found on the FHWA Office of Operations website, and the AASHTO *Guide for the Design of High Occupancy Vehicle Facilities*.

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11.8 BUS STOP AND TURNOUTS

11.8.1 Location

11.8.1.1 Bus Stops

If local bus routes are located on an urban or suburban highway, the designer should consider their impact on normal traffic operations. The stop-and-go pattern of local buses will disrupt traffic flow, but certain measures can minimize the disruption. The location of bus stops is particularly important. These are determined not only by convenience to patrons, but also by the design and operational characteristics of the highway and the roadside environment.

There are three basic bus stop designs — far-side or near-side of an intersection, and mid-block. These designs are shown in Figure 11.8-A and discussed below:

1. Far-Side Stop. Typically, far-side intersection placement is desirable. Placing turnouts after signal-controlled intersections allows the signal to create gaps in traffic.

2. Near-Side Stop. Avoid using near-side turnouts because of conflicts with right-turning vehicles, delays to transit services as buses try to re-enter the traveled way, and obstructions to traffic control devices and pedestrian activities.

3. Mid-Block Stop. Only use mid-block turnouts in conjunction with major traffic generators.

11.8.1.2 Selection

In general, far side locations of bus stops and turnouts are preferred. The municipality or local transit authority will determine the location of the bus stop or bus turnout. However, the designer usually has some control over the best placement of a bus stop or turnout location when considering layout details, intersection design and traffic flow patterns.

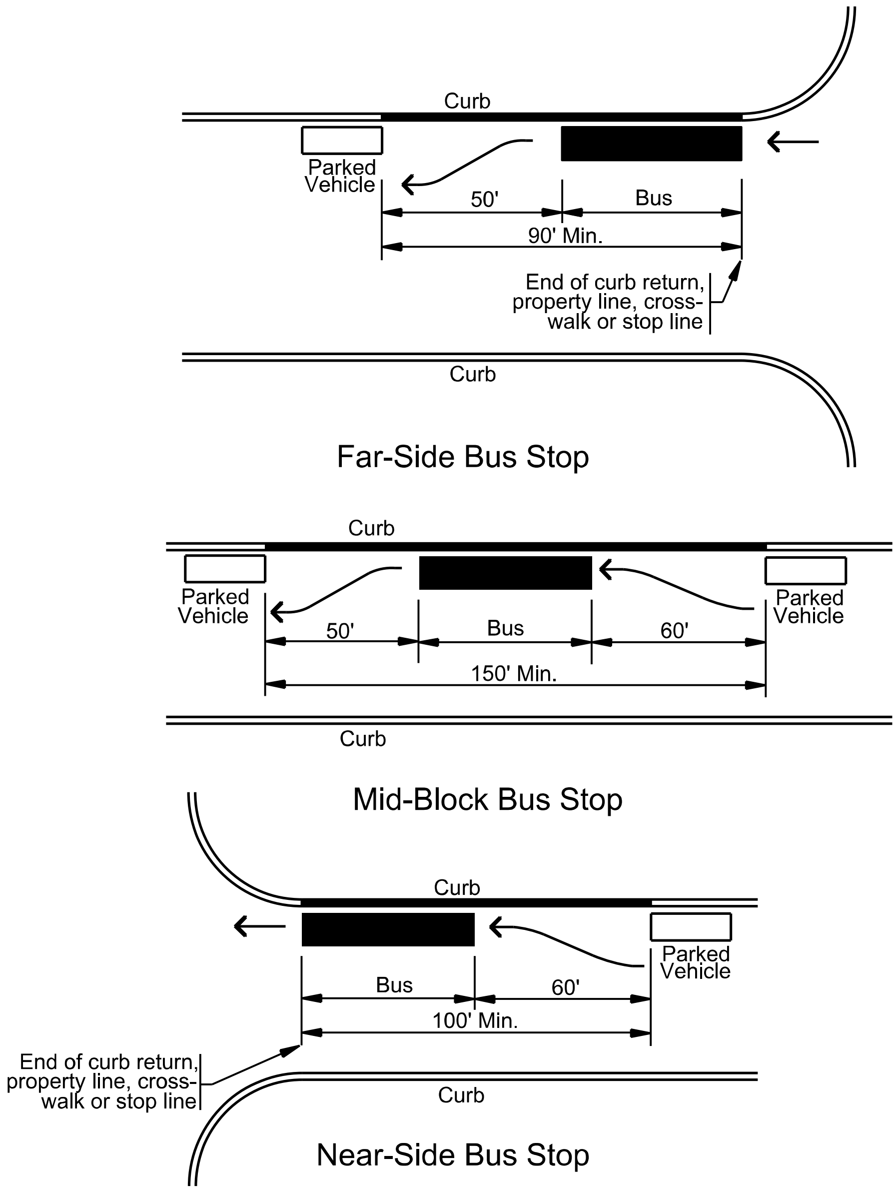
11.8.2 Design

11.8.2.1 Bus Stops

Figure 11.8-A provides the recommended distances for the prohibition of on-street parking near bus stops.

11.8.2.2 Bus Stop Pads

All new bus stops that are constructed for use with lifts or ramps must meet the accessibility criteria in the Department's ADA Transition Plan.



Notes:

1. *Where articulated buses are expected, use a bus length of 60 feet.*
2. *Provide an additional 50 feet of length for each additional bus expected to stop simultaneously at any given bus stop area. This allows for the length of the extra bus, 40 feet, plus 10 feet between buses.*

RECOMMENDED PARKING RESTRICTIONS FOR ON-STREET BUS STOPS
Figure 11.8-A

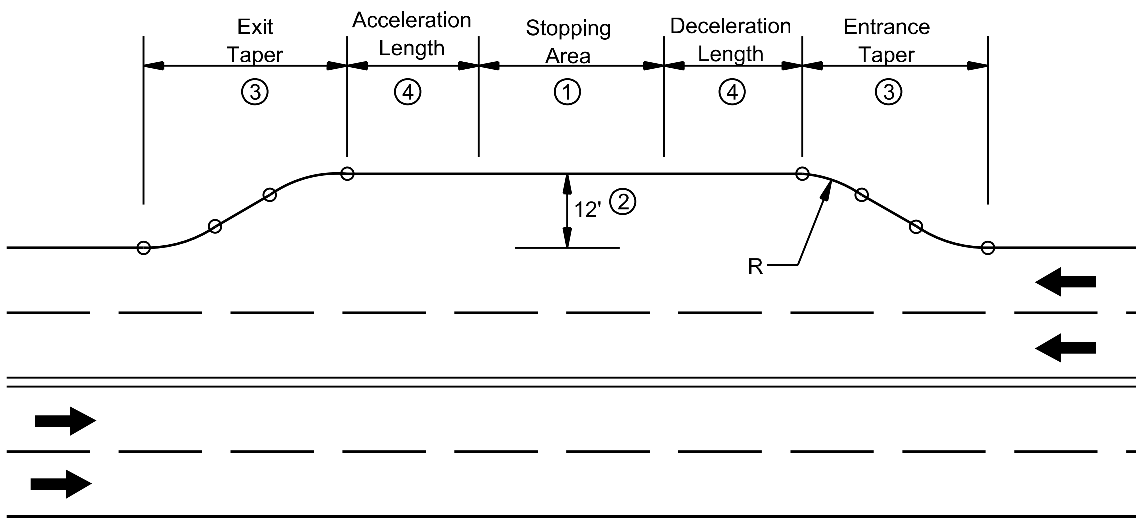
11.8.2.3 Bus Turnouts

Desirably, the total length of a bus turnout will allow for an entrance taper, a deceleration length, a stopping area, an acceleration length and an exit taper. Providing bus turnouts can reduce interference between buses and other traffic significantly. Turnouts remove stopped buses from the through lanes and provide a well-defined user area for bus stops. Figure 11.8-B illustrates the design details for bus turnouts. Providing separate deceleration and acceleration lengths are desirable in suburban areas and on rural arterials and may be provided wherever feasible. However, common practice is to accept deceleration and acceleration in the through lanes and only constructing the tapers and stopping area.

Figure 11.8-B provides information on taper lengths that may be used for entrance and exit tapers. To improve traffic operations, use short horizontal curves (100-foot radius) on the entry end and 50-foot to 100-foot curves on the re-entry end. Where a turnout is located at a far-side or near-side location, the cross street area can be assumed to fulfill the need for the exit or entry area, whichever applies.

11.8.2.4 Bus Shelters

In general, the municipality or the local transit authority will determine the need for and location of bus shelters. The local transit authority will determine the design of the bus shelter. The designer should ensure that the shelter does not restrict vehicular sight distance, pedestrian flow or pedestrian accessibility.



Notes:

① *Stopping area length consists of the length of the bus plus 10 feet for each bus expected to be at the stop simultaneously.*

② *Bus turnout width is desirably 12 feet. For posted speeds under 30 miles per hour, a 10-foot minimum bay width is acceptable. These dimensions do not include the gutter width.*

③ *Suggested taper lengths are listed below. A minimum taper of 5:1 may be used for an entrance taper from an arterial street for a bus turnout while the exit or re-entry taper should not be sharper than 3:1.*

④ *The minimum design for a bus turnout does not include acceleration or deceleration lengths. Recommended acceleration and deceleration lengths are listed below.*

Design Speed	Entering Speed*	Acceleration Lengths	Deceleration Lengths **	Suggested Taper Lengths
35 mph	25 mph	250 ft	185 ft	170 ft
40 mph	30 mph	400 ft	265 ft	190 ft
45 mph	35 mph	700 ft	360 ft	210 ft
50 mph	40 mph	975 ft	470 ft	230 ft

* *Desirably, the bus speed at the end of taper should be within 10 miles per hour of the design speed of the traveled way.*

** *Based on a 2.5 miles per hour per second deceleration rate.*

TYPICAL BUS TURNOUT DIMENSIONS
Figure 11.8-B

11.9 MAILBOXES

Mailboxes should be placed with safety considerations for motorists, pedestrians, the carrier and the postal patron. The designer should consider the walking distance for the patron, pedestrian access route, stopping sight distance in advance of the mailbox and sight distance restrictions at intersections and driveways. For additional information on the placement of mailboxes on roadways, see the AASHTO *Roadside Design Guide*.

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11.10 CUL-DE-SACS

A local street open at only one end should have a special turning area at the closed end. The commonly used design is a circular pavement symmetrical about the centerline of the street sometimes with a central island. Provide minimum outside radii of 30 feet in residential areas and 50 feet in commercial and industrial areas; see the SCDOT *Standard Drawings*. Improved operations may be obtained if the design is offset so that the entrance of the pavement is in line with the approach half of the street. One steering reversal is avoided with this design. The designer should revise the geometry of the cul-de-sac if adjoining residences also use the cul-de-sac for parking. The designer should give consideration to through pedestrian traffic when a cul-de-sac is constructed on an existing through street.

Other variation or shapes of cul-de-sacs that include right-of-way and site controls may be provided to permit vehicles to turn around by backing only once. Alternative designs are discussed in the AASHTO *A Policy on Geometric Design of Highways and Streets*.

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11.11 BICYCLE ACCOMMODATIONS

Provisions for bicycles are an important consideration where new highways are being constructed or existing highways are being widened or otherwise improved. This is particularly true in urban and suburban areas and where tourism, congestion mitigation and alternative modes of transportation are major factors. In rural areas, bicycling accommodations will typically be on the roadway shoulder. In urban areas, bicycling accommodations may be provided by a shared roadway or dedicated space (e.g., designated bicycle lanes). Consider separate bicycle facilities where bicyclists would become involved with high-traffic volume roadways. For pedestrian safety, the designer should not consider sidewalks as bicycle facilities.

11.11.1 Bikeway Classifications

The following definitions apply to bikeway classifications:

1. Bicycle Boulevard. A street segment, or series of contiguous street segments, that has been modified to accommodate through bicycle traffic, but discourages through motor traffic with traffic calming measures.

2. Bicycle Lane or Bike Lane. A portion of a roadway that has been designated by striping, signing and pavement markings for the preferential or exclusive use of bicyclists. It is distinguished from the travel portion of the roadway by a physical or symbolic barrier.

3. Bikeway. A pathway that is exclusively used by bicyclists, where a separate, parallel path is provided for pedestrians and other wheeled users. Most pathways are shared between bicyclists and other users; see shared use path.

4. Bicycle Route. A roadway or bikeway designated by the jurisdiction having authority, either with a unique route designation or with BIKE ROUTE signs, along which bicycle guide signs may provide directional and distance information. Signs that provide directional, distance and destination information for cyclists do not necessarily establish a bicycle route.

5. Shared Roadway. A roadway that is open to both bicycle and motor vehicle travel. This may be an existing roadway, a street with wide curb lanes or a road with paved shoulders.

6. Shared Use Path. A path physically separated from motorized vehicular traffic by an open space or barrier and either within the highway right of way or within an independent right of way. Shared use paths may also be used by pedestrians, skaters, wheelchair users, joggers and other non-motorized users.

7. Shared Lane. A lane of a traveled way that is open to bicycle travel and vehicular use.

8. Sidepath. A shared use path located immediately adjacent and parallel to a roadway.

9. Rail-Trail. A shared use path, either paved or unpaved, built within the right of way of a former railroad.

10. Rail-with-Trail. A shared use path, either paved or unpaved, built within the right of way of an active railroad.

11.11.2 Selection Guidelines

Although incorporating bicyclists' needs into the design of major transportation corridors can be challenging, the reality of planning bikeways in built environments means that roadways constitute the majority of a bicycle network. Whenever streets are constructed or reconstructed, the designer should include appropriate provisions for bicyclists. The designer needs to check the local, regional or Statewide approved bicycle plans. Bicycle accommodations options include:

- paved shoulders,
- bike lanes,
- bike boulevards,
- shared lanes,
- shared roadways, and
- shared use paths.

Selection of an appropriate bikeway facility requires the following information:

- road function (arterial, collector, local);
- vehicular and bike volume;
- speed;
- traffic mix (e.g., truck %);
- expected users (e.g., one type of user expected to dominate, such as children bicycling to school);
- road conditions (lane widths, total roadway width, conditions at intersections, parking demand);
- frequency of driveways and side streets;
- topography;
- existing and proposed adjacent land uses; and
- approved bikeway and transportation plan.

Figure 11.11-A outlines general considerations for each facility type.

Type Of Bikeway	Best Use	Motor Vehicle Design Speed	Traffic Volume	Classification or Intended Use	Other Considerations
Paved shoulders	Rural highways that connect town centers and other major attractors	Variable. Typical posted rural highway speeds (generally 40 mph – 55 mph)	Variable	Rural roadways; inter-city highways	Provides more shoulder width for roadway stability. Shoulder width is dependent on characteristics of the adjacent motor vehicle traffic (e.g., wider shoulders on higher-speed roads).
Bike lanes	Major roads that provide direct, convenient, quick access to major land uses. Also can be used on collector roads and busy urban streets with slower speeds.	Generally, any road where the design speed is more than 25 mph.	Variable. Speed differential is generally a more important factor in the decision to provide bike lanes than traffic volumes.	Arterials and collectors intended for major motor vehicle traffic movements	Where motor vehicles are allowed to park adjacent to bike lane, ensure width of bike lane sufficient to reduce probability of conflicts due to opening vehicle doors and other hazards. Analyze intersections to reduce bicyclist/motor vehicle conflicts. Sometimes bike lanes are left “undesignated” (i.e., bicycle symbol and signs are not used) in urban areas as an interim measure.
Bike boulevard	Local roads with low volumes and speeds, offering an alternative to, but running parallel to, major roads. Still should offer convenient access to land use destinations.	Use where the speed differential between motorists and bicyclists is typically 15 mph or less. Generally, posted speed limits of 25 mph or less.	Generally less than 3000 vehicles per day	Residential roadways	Typically only an option for gridded street networks. Avoid requiring bicyclists to make frequent stops. Use signs, diverters and other treatments so that motor vehicle traffic is not attracted from arterials to bike boulevards.
Shared lanes (wide outside lanes)	Major roads where bike lanes are not selected due to space constraints or other limitations.	Variable. Use as the speed differential between bicyclists and motorists increases. Generally, any road where the design speed is more than 25 mph.	Generally more than 3000 vehicles per day	Arterials and collectors intended for major motor vehicle traffic movements	Explore opportunities to provide parallel facilities for less confident bicyclists.

GENERAL CONSIDERATIONS FOR BIKEWAYS
Figure 11.11-A

Type of Bikeway	Best Use	Motor Vehicle Design Speed	Traffic Volume	Classification or Intended Use	Other Considerations
Shared roadways (no special provisions)	Minor roads with low speeds and volumes, where bicycles can share the road with no special provisions.	Speed differential between motorists and bicyclists is typically 15 mph or less. Generally, speed limits of 30 mph or less.	Generally less than 1000 vehicles per day	Neighborhood or local streets	Can provide an alternative to busier streets in a gridded street network. On a non-grid network, may be circuitous or discontinuous.
Shared use path: adjacent to roadways (i.e., sidepath)	Adjacent to roadways with no or very few intersections or driveways. The path is used for a short distance to provide continuity between sections of path on independent right-of-way.	The adjacent roadway has high-speed motor vehicle traffic such that bicyclists might be discouraged from riding on the roadway.	The adjacent roadway has very high motor vehicle traffic volumes such that bicyclists might be discouraged from riding on the roadway.	Provides a separated path for non-motorized users. Intended to supplement a network of on-road bike lanes, shared lanes, bicycle boulevards and paved shoulders. Not intended to substitute or replace on-road accommodations for bicyclists, unless bicycle use is prohibited.	Several operational issues are associated with this facility type. See AASHTO *Guide for the Development of Bicycle Facilities*.
Shared use path: independent corridor	Linear corridors in green ways, or along waterways, highways, active or abandoned rail lines, utility right of way, unused right of way. May be a short connection, (e.g., pathway connector between two cul-de-sacs) or a longer connection.	n/a	n/a	Provides a separated path for non-motorized users.	Analyze intersections to anticipate and mitigate conflicts between path and roadway users. Design the path with all users in mind, wide enough to accommodate expected usage. On-road alternatives may be desired for advanced riders who desire a more direct facility that accommodates higher speeds.

GENERAL CONSIDERATIONS FOR BIKEWAYS
Figure 11.11-A
(Continued)

11.11.3 Design Criteria

For the design criteria of bicycle facilities, the designer should refer to the AASHTO *Guide for the Development of Bicycle Facilities*. The AASHTO Guide contains criteria on the design of bicycle facilities which includes, for example, railroad crossings, intersections, criteria for horizontal/vertical alignment, pavements, traffic control devices, etc. In addition to the AASHTO Guide, the designer should review the criteria in the following sections.

11.11.3.1 Widths

Figures 11.11-B through 11.11-D provide typical cross sections for new construction roadways with bicycle accommodations. In addition, the designer should consider the following guidance for bicycle accommodations:

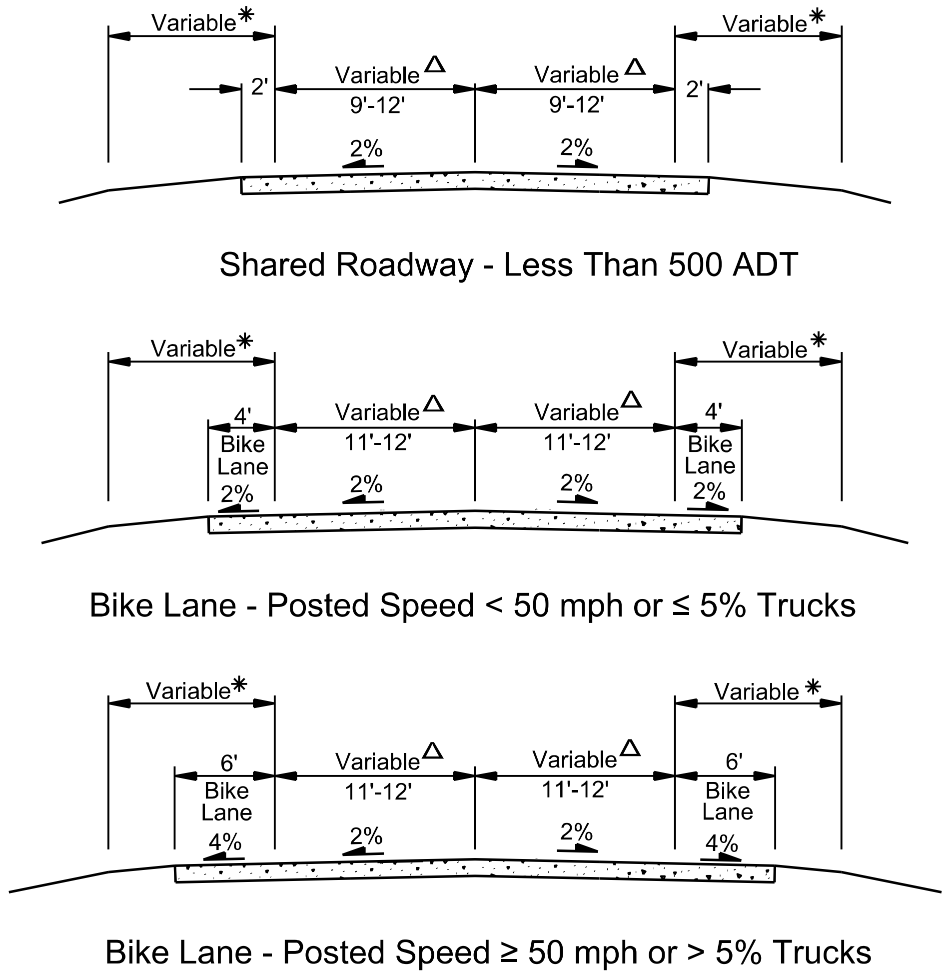
1. Shared Lanes. On urban sections (curb and gutter), an outside travel lane width of 14 feet is the minimum recommended width for a shared-use lane. Lane widths that are 14 feet or greater allow motorist to pass bicyclist without encroaching into the adjacent lane. The usable lane width is normally measured from the center of the edge line to the center of the traffic lane line, or from the longitudinal joint of the gutter pan to the center of the lane line. The gutter pan is not included in the width of the shared lane.

   On sections of roadways where bicyclists may need more maneuvering space, consider providing a 15-foot travel lane width. This width may be appropriate on sections with steep grades (greater than 5 percent) or where drainage grates, raised delineators or on-street parking effectively reduce the usable width. Shared lane widths greater than 16 feet that extend continuously along stretches of roadway may encourage undesirable motor vehicle operations, especially in urban areas (e.g., two motor vehicles to travel side by side, faster vehicular speeds). Therefore, shared lanes greater than 16 feet are not recommended and consideration should be given to striping the additional width.

   Roadways with shared lanes narrower than 14 feet may still be designated for bicycles with bicycle guide signs and/or shared-lane markings.

2. Paved Shoulders. On rural sections (shoulder) with ADT greater than 500, paved shoulders should be a minimum of 4 feet wide in each direction to accommodate bicycle travel. Where motor vehicle speeds exceed 50 miles per hour or the percentage of trucks, buses and recreational vehicles is greater than 5 percent of the ADT, the designer should give consideration to providing a minimum width of 6 feet to accommodate bicycle travel adjacent to the higher speeds (50 miles per hour or greater) and to lessen the effect of windblast from larger vehicles. On rural sections (shoulder) with ADT less than 500, paving 2 feet of the earthen shoulder typically will be adequate to better accommodate bicyclists.

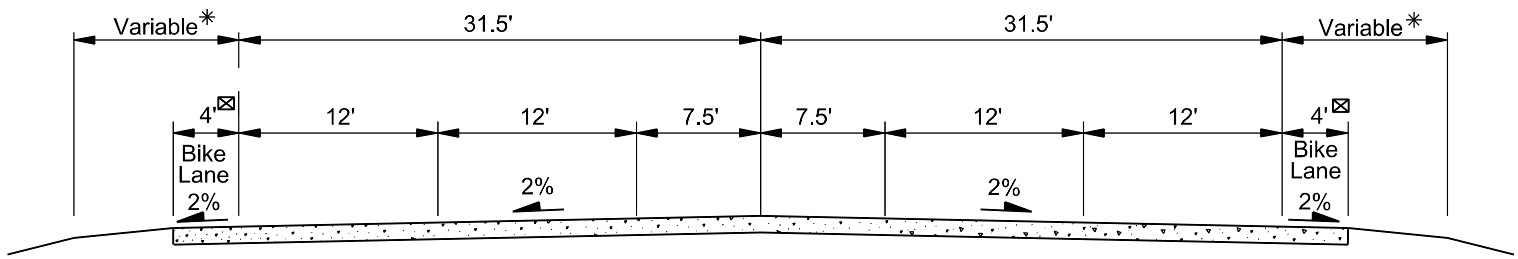
   On urban sections (curb and gutter), paved shoulders should be a minimum of 4 feet wide to accommodate bicycle travel and have a cross slope of 50H:1V (2.00 percent). Do not include the gutter pan in the width of the paved shoulder. Where the percentage of trucks, buses and recreational vehicles is greater than 5 percent of the ADT, consider providing a minimum width of 6 feet. Where motor vehicle speeds are



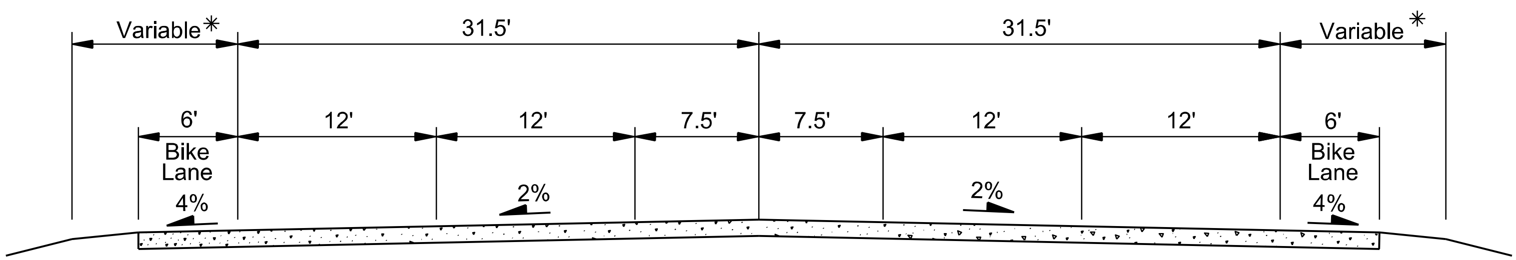
* Shoulder Width Per Chapters 14, 15 and 16

Δ Lane Widths Per Chapters 14, 15 and 16

BICYCLE FACILITIES
(New Construction – Two-Lane Rural Section with Paved Shoulders)
Figure 11.11-B



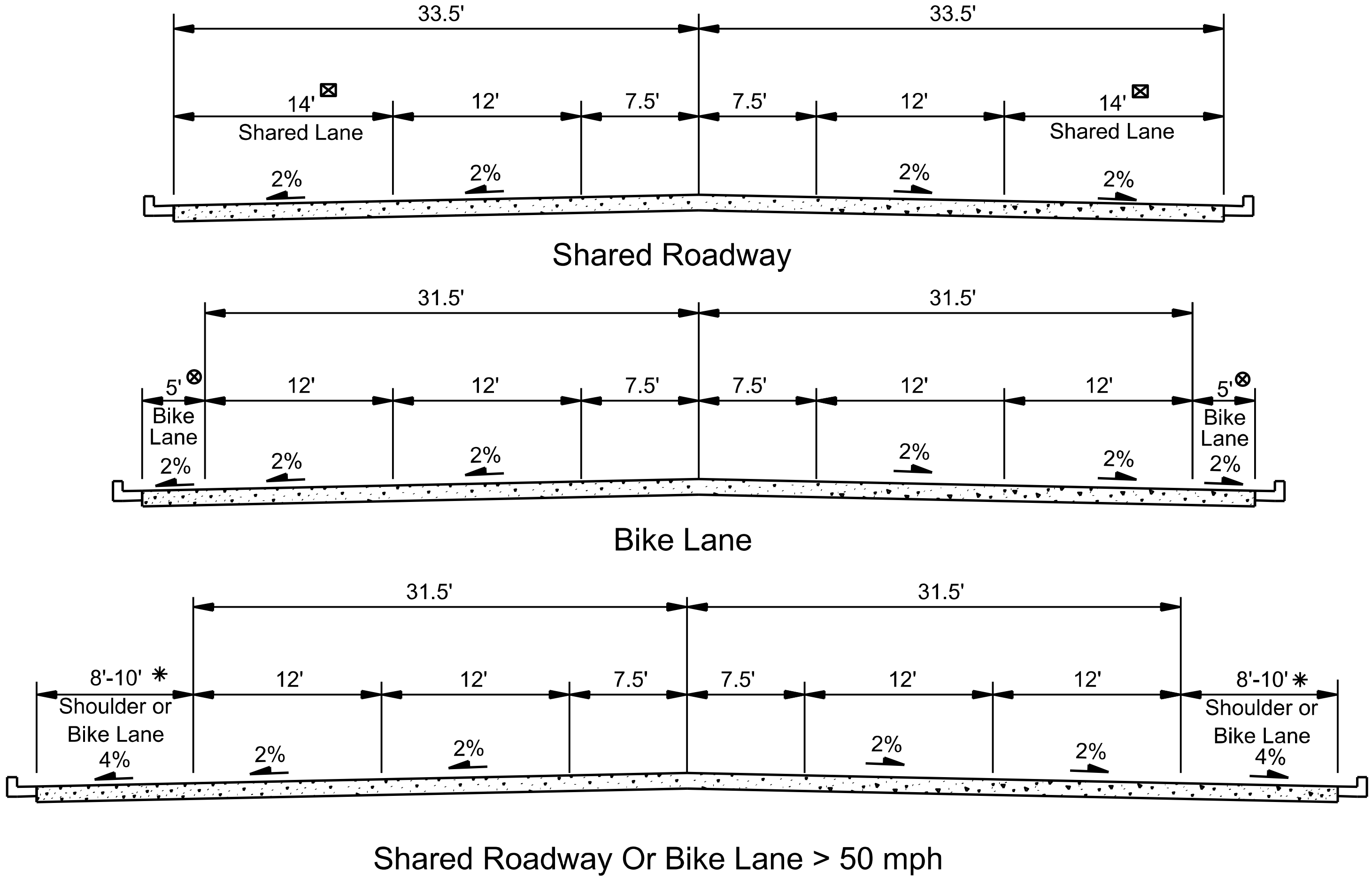
Bike Lane - Posted Speed < 50 mph or ≤ 5% Trucks



Bike Lane - Posted Speed ≥ 50 mph or > 5% Trucks

✳ Shoulder Width Per Chapters 14, 15 and 16
⊠ Use a 2' Paved Shoulder for a Shared Roadway

BICYCLE FACILITIES
(New Construction – Five-Lane Rural Section with Paved Shoulders)
Figure 11.11-C



✳ Shoulder Width Per Chapters 14, 15 and 16
⊠ Consider Using 15' When Grades > 5%
⊗ Consider Using 6' When > 5% Trucks

BICYCLE FACILITIES
(New Construction – Five-Lane Urban Section with Curb and Gutter)
Figure 11.11-D

50 miles per hour or greater, use the guidelines for roadway shoulder widths provided in Chapters 14 “Local Roads and Streets,” 15 “Collector Roads and Streets” and 16 “Rural and Urban Arterials” (i.e., 8 or 10 feet of paved shoulder).

3. Bicycle Lanes. On two-way streets, bicycle lanes should be provided on both sides of the street. The typical bike lane width is 4 feet. The designer should consider wider widths under the following conditions:

   a. Adjacent to a narrow (7 feet) parking lane with high turnover (e.g., those servicing restaurants, shops or entertainment venues), a wider bicycle lane (6 feet to 7 feet) provides more operating space for bicyclists to ride out of the area of opening vehicle doors.

   b. In areas with high bicycle use, a bike lane width of 6 feet to 8 feet makes it possible for bicyclists to ride side-by-side or pass each other without leaving the lane.

   c. On high speed (45 miles per hour or greater) and high-volume roadways, or where there is a substantial volume of heavy vehicles, a wide bicycle lane provides additional lateral separation between motor vehicles and bicycles to minimize wind blast and other effects.

4. Shared Use Paths. The minimum paved width for a two-directional shared use path is 10 feet, with typical widths ranging from 10 feet to 14 feet. Wider widths are recommended where there are a high percentage of pedestrians (30 percent or more of the total users) or where there are more than 300 total users in the peak hour.

5. One-Way Streets. For a bike lane to function as intended when built against the dominant flow of traffic on a one-way street, incorporate the following features into the design:

   a. Bike Lane Location. Place the bike lane on the right side of the roadway.

   b. Narrow Streets. A bike lane should be provided for bicyclists traveling in the same direction as motor vehicle traffic. If there is insufficient room to provide a bike lane in the dominant-flow direction of the street, consider providing shared-lane markings to emphasize that bicyclists must share the travel lane on this side of the street.

   c. Contraflow Bike Lanes. Contraflow bike lanes require special considerations and should only be considered after analysis of bike patterns. If contraflow bike lanes are to be included in the design, see the guidance on contraflow bike lanes provided in the AASHTO *Guide for the Development of Bicycle Facilities*.

   d. Intersections. Use bike lane symbols and directional arrows on both the approach and departure of each intersection, to remind bicyclists to use the bike lane in the appropriate direction, and to remind motorists to expect bicycle traffic.

11.11.3.2 Paving Existing Shoulders

In order for a shoulder to be usable to a bicyclist, it generally must be paved. Adding or improving paved shoulders often can be the best way to accommodate bicyclists in rural areas and benefit motor vehicle traffic. Paved shoulders have the added benefit of not only accommodating bicyclists, but they can also extend the service life of the road surface (i.e., edge deterioration will be significantly reduced). Provide 2 feet of paved shoulder width on all new projects with earthen shoulders. Where practical and attainable, provide a minimum paved shoulder width of 4 feet to provide for bicycle facilities where the AADT of the road is greater than 500. Where constraints do not allow obtaining the indicated widths, any additional width will be beneficial to a bicyclist.

11.11.3.3 Resurfacing/Restriping Existing Roadways

Where it is desirable to accommodate bicycle facilities by resurfacing/restriping existing roadways, lane and/or median widths may be narrowed to obtain the desired bicycle facility. Roadways designated as being on the National Truck Network or South Carolina Truck Network or roadways where the percentage of trucks, buses and recreational vehicles is greater than 5 percent of the AADT should have lane widths of 12 feet. Where conditions allow using lane widths narrower than 12 feet to accommodate bicycle facilities, the designer should determine the impacts of narrower lane widths to motor vehicle traffic. Guidance on selecting the proper lane width for a roadway can be found in Chapters 14 through 16.

The typical flush/painted median width is 15 feet; however, the width can be reduced to 12 feet to accommodate bicycle facilities on an existing roadway or existing project; see Figure 11.8-E. Median widths less than 12 feet are not recommended where posted speeds are greater than 35 miles per hour and the percentage of trucks, buses and recreational vehicles is greater than 5 percent of the ADT.

11.11.3.4 Drainage Inlet Grates

Where practical, place all drainage inlets outside of the bicycle facility. Where this is not practical, use hydraulically efficient bicycle-safe grates and place or adjust the inlets to be flush with the adjacent pavement surface. On bridges, a minimum of 4 feet from the edge of the travel lane should be clear of drainage inlets.

11.11.3.5 Longitudinal Rumble Strips

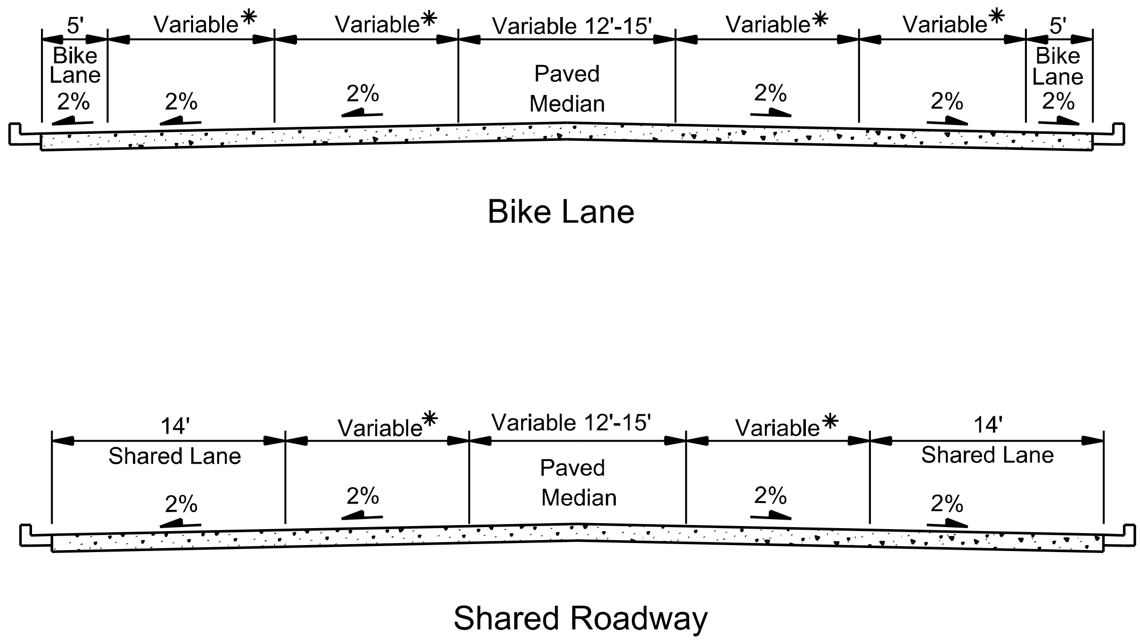
See Engineering Directive 53 “Installation of Rumble Strips” for guidance on rumble strips and bicyclists.

11.11.3.6 Bridges

Generally, bridge widths should match the approach roadway widths (traveled way plus bike lanes/shoulders). However, in determining the width for major water crossings, consider the cost of the structure, traffic volume, structural design life and potential for future width requirements.

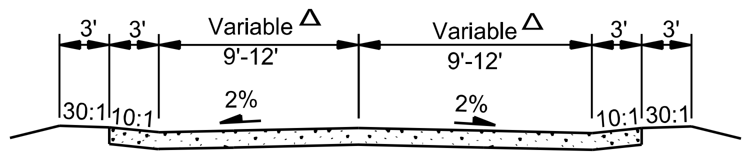
11.11.3.7 Valley Gutter Sections

See Figure 11.8-F for guidance on shared roadways and bike lanes/paved shoulders widths adjacent to valley gutters. Because valley gutter sections are typically used on low-volume, two-lane local roadways, the cross slope of the paved shoulder/bike lane should be 50H:1V (2.00 percent).

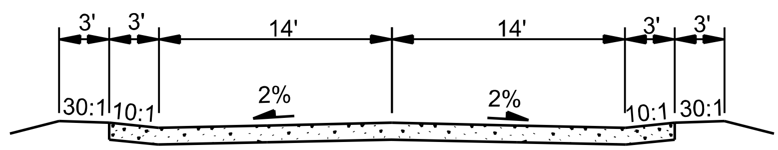


*11'-12' Lane Widths
(On National or South Carolina Truck Network Use 12' Min. Lane Width)

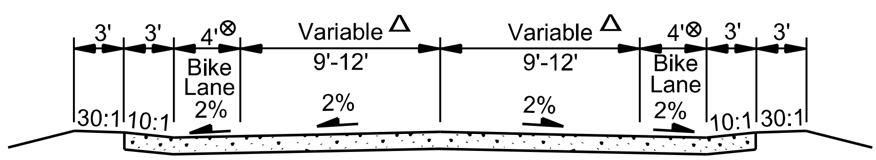
BICYCLE FACILITIES
(Restriping Existing Five-Lane Urban Section with Curb and Gutter)
Figure 11.11-E



Shared Roadway - Less Than 500 ADT



Shared Roadway



Bike Lane

△ Lane Widths Per Chapter 14
⊗ Consider Using 6' When > 5% Trucks

BICYCLE FACILITIES
(New Construction – Two Lane with Valley Gutter Sections)
Figure 11.11-F

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11.12 REFERENCES

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3. *SCDOT Bridge Design Manual*, SCDOT, 2006.
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Chapter 12

RIGHT OF WAY

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 12
RIGHT OF WAY

The purpose of this chapter is to provide an overall understanding of right-of-way elements and their impact on the design of roadway projects. This information will provide the designer with an overview of SCDOT policies and procedures relative to right of way. This chapter will allow right-of-way issues to be properly addressed during design and right-of-way information to be completely documented on the Right of Way Plans. Specific information, relative to the provisions of right of way, is contained in the *SCDOT Right of Way Acquisition Manual* and the Department's guidelines for the preparation of right-of-way exhibits.

The term Right of Way Plans refers to the stage at which roadway and bridge plans are adequately developed for the acquisition of right of way. This includes, at a minimum, an accurate representation of existing topographic and property features and all items that affect the final establishment of new right-of-way limits. Sufficient drainage design should also be included to show the extent of all ditch work, including all sediment and erosion control measures.

12.1 RIGHT-OF-WAY DESIGN

It is important that the designer be familiar with the rules and policies for establishing and depicting right of way and related data on the plans. Consistency in this area permits easier reviews by Rights of Way Office and aids the appraisal process and preparation of instruments of record.

12.1.1 General Right-of-Way Information

New right-of-way boundaries should represent the limits of usable area required for construction, maintenance activities following construction, drainage (e.g., outfall ditches) and traffic control devices.

In rural areas, maintenance activities (e.g., mowing, ditch and channel cleanout, shoulder repairs) are more easily accomplished within an adequate right of way. An important reason for providing wider right of way is for adequate clear zones and to provide sufficient sight distance at intersections and around curves. In general, the preliminary right-of-way limits should be established a sufficient distance outside the construction limits to accommodate future maintenance activities and adjusted to tighter limits as significant conflicts dictate. The designer should adjust the new right-of-way limits for uniformity, to eliminate transition lines and to achieve a uniform offset from the survey centerline for extended lengths, as practical. These considerations are generally not applicable in urban areas due to high property costs and because property owners often maintain grounds adjacent to, and often within, the right of way.

When preliminary road design plan drawings are complete, a field review may be conducted. In these cases, the plans may only show the existing right of way. After the field review is complete, the designer is responsible for adding the proposed new right of way to the plans.

12.1.2 Present Right of Way

See Section 22.2.6.1.1 for information on verifying present right of way and noting it on the plan sheets.

12.1.3 Right-of-Way Plats and Monuments

For projects with new right of way, the contractor is required to place right-of-way monuments in the field and develop the applicable right-of-way plats. The designer is responsible for ensuring the pay items for right-of-way plats and monuments are included as bid items on all construction projects that require new right of way.

See the *SCDOT Standard Drawings* for details and placement of the markers.

12.1.4 Right-of-Way Widths

New right-of-way boundaries may be of uniform widths, varying widths or a combination thereof. If there appears to be enough uniformity throughout the project limits, it is desirable to establish a uniform width and apply it to the design throughout. Any work required outside the uniform width may be accomplished by obtaining the property owner's permission. This procedure will allow the construction to take place outside of the right of way and when construction activity is completed, the Department has no further rights or obligations.

Determining the width of the permanent right of way is primarily a function of the typical section, along with safety, drainage requirements and requirements of the National Pollutant Discharge Elimination System (NPDES); see Section 12.1.5. Evaluate the new right-of-way limits during the field review.

Desirably, the new right-of-way line should be established sufficiently beyond the construction lines (10 feet to 15 feet) to permit maintenance equipment to work outside of and parallel to the construction limits, particularly for freeways, expressways or other major highways. In urban and suburban areas, this is usually not practical due to the high cost of property. In congested urban areas, the right-of-way line is set a minimum of 0.5 foot from the back edge of the sidewalk and permission is obtained for slopes that extend beyond these limits.

12.1.5 National Pollutant Discharge Elimination System (NPDES)

12.1.5.1 Setting the Location of Right-of-Way and Permissions for NPDES Requirements

Land disturbing activities determined necessary for the construction and maintenance of projects may require additional right of way or permissions. Secure permanent right of way for all land disturbing activities to be maintained after completion of a project and around all sediment control basins (temporary and permanent). Secure permission for all other temporary land disturbing activities (e.g., cleaning outfall ditches). If permission cannot be obtained, then the area will be acquired as right of way. In both instances, the area will be cleared and grubbed and seeded during construction. Consideration can be given to eliminating grubbing and providing only clearing in areas with jurisdictional boundaries.

Where additional right of way is more difficult to obtain due to high cost, urban areas, wetlands and/or significant trees, consider all means to circumvent these conflicts by minimizing the need for additional right of way, while still allowing implementation and maintenance of necessary erosion control facilities. Ensure the design plans address the general and special conditions of the environmental permit to minimize impacts.

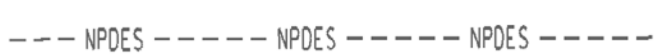
An area between the silt fence and the toe of slope is needed to properly maintain the silt fence. Large equipment and trucks may use the area in front of the silt fence to access and remove of any sediment collected by the silt fence or a nearby silt basin. It is expected that the area between the silt fence and the toe of the slope is cleared and grubbed during construction and maintained with temporary seeding. When this additional area in front of the silt fence cannot be obtained, the maintenance of the silt fence will be handled as best as practical during construction.

Right-of-way limits in cut slope areas should be determined during the Design Field Review where interceptor ditches or other erosion control items are deemed necessary. The right-of-way line should maintain a uniform alignment and not fluctuate in and out, where practical. The designer should use discretion when establishing right-of-way boundaries in order to minimize areas not needed for the construction and maintenance of the project.

12.1.5.2 Denoting NPDES Requirements on Plans

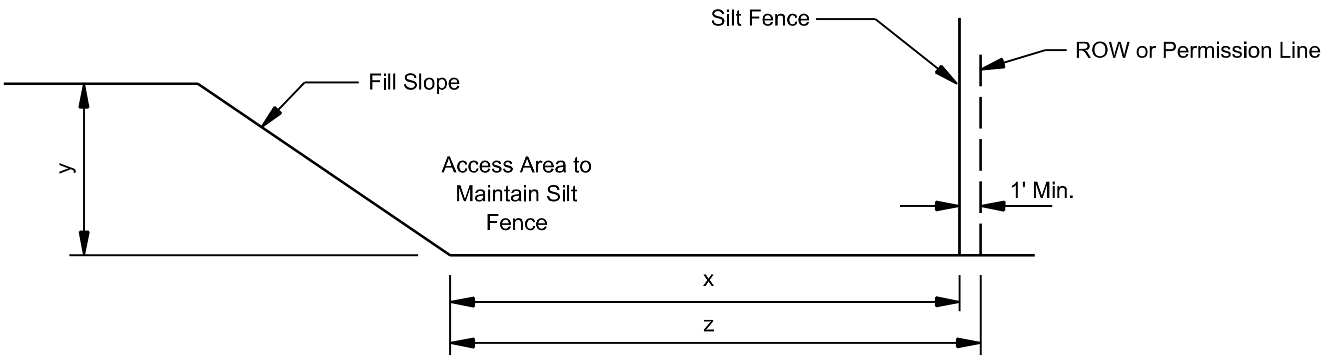
Place a special line denoting land disturbing activities for NPDES only when it necessary to go beyond the construction limits. The NPDES line (see Figure 12.1-A) will have offset distances from the construction limits as specified in Figure 12.1-B. In areas where temporary land disturbing activities are being performed and no construction limits are present (e.g., cleaning outfall ditches and bridge construction access), place the NPDES line around disturbed area. This special line can be found in the custom line style palette and is shown in Figure 12.1-A.

Provide a silt fence for all fill slopes in order to minimize the erosion of sediment off the project site. Place the silt fences beyond the toe of the fill slope as shown in Figure 12.1-B. All silt fences must be cleared periodically as sediment is collected. The anticipated reach of the contractor’s equipment can be assumed to be 15 feet.



NPDES LINE
Figure 12.1-A

Height of Fill (y) (feet)	Fill Slope	Minimum Silt Fence Offset from Toe of Slope (x) (feet)	NPDES Line Location Offset from Toe of Slope (z) (feet)
<6	2H:1V 4H:1V 6H:1V	2	3
6-10	2H:1V	12	13
	4H:1V	3	4
	6H:1V	3	4
>10	2H:1V	12	13
	4H:1V	4	5
	6H:1V	4	5



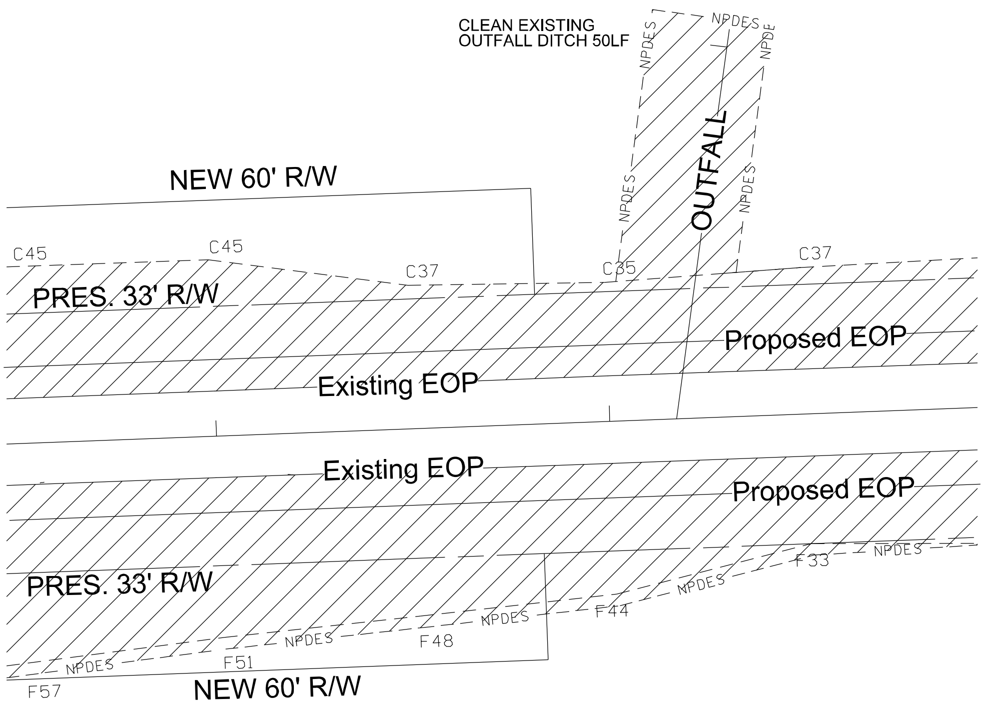
SILT FENCE OFFSETS
Figure 12.1-B

12.1.5.3 Computing NPDES Acreage

Compute the Disturbed Area for NPDES by determining the area between the NPDES lines less any existing pavement to be retained. The traveled way on existing dirt roads will not be included in the Disturbed Area. Use the construction limits (cuts/fills) in lieu of NPDES lines in areas where NPDES lines are not shown. See Figure 12.1-C for an example of Disturbed Area. Show the Disturbed Area in acres, rounded up to the nearest tenth of an acre, on the Title Sheet within the NPDES Permit Information box.

Compute the Project Area for NPDES by adding up to 2 acres to the calculated Disturbed Area for projects less than 20 acres in size or by adding up to 3 acres to the calculated Disturbed Areas for projects 20 acres and greater in size. Show the Project Area in acres, rounded up to the nearest tenth of an acre, on the Title Sheet within the NPDES Permit Information box.

All increases in Disturbed Area must be approved by the Resident Engineer or their designee and noted on the marked-up plan sheets. Increases in disturbed area can be no more than 1 acre at any one location. Any Disturbed Area increases, whether one-time or the sum of previous increases, causing the Disturbed Area to surpass the Project Area will be considered a Major Modification. The Notice of Intent (NOI) must be updated and resubmitted to SCDHEC for all Major Modifications. Work in locations that create a need for a Major Modification cannot commence until approval is received from SCHDHEC.



DISTURBED AREAS
Figure 12.1-C

12.1.5.4 NPDES Quantities and Bid Items

Temporary NPDES facilities installed by permission will be seeded, according to the temporary seeding schedule, at the time of installation. Use the permanent seeding schedule after the temporary NPDES facility has been removed and the area reclaimed. Permanent NPDES facilities will be seeded according to the normal seeding schedule. All seeding will be completed and paid for in accordance with the *SCDOT Standard Specifications for Highway Construction*.

If the area required for NPDES is to be reclaimed, then include the quantity of soil for re-grading in the total quantity of "Silt Basins" and show the necessary seeding in the quantities. The following items are to be removed and disposed of in the bid item "Temporary Sediment Control Structure" where it is necessary to reclaim the area in which a "Temporary Sediment Control Structure and Basin" is located:

- the structure and appurtenances,
- all riprap associated with that basin,
- pipe connected to the structure,
- anti-seep collars, and
- fence and gate surrounding the basin.

12.1.5.5 Coordination of Hydrology/NPDES Studies

It is always preferable to have the final hydraulic and NPDES designs shown on the plans for right-of-way acquisition. When the final hydraulic/NPDES designs are not available to be placed on the Right of Way Plans, make every effort to include all hydraulic/NPDES designs that affect right of way. However, when Right of Way Plans have been sent to the Rights of Way Office prior to receiving the final hydraulic and NPDES studies, revisions to the Plans, especially to the existing hydrology and erosion control elements, can be expected. Upon receipt of the final hydraulic and NPDES design from the hydraulic designer, the roadway designer will make the necessary revisions, noting appropriately on each sheet where the following revisions are made: "Revisions made to Tract XX in accordance with the hydraulic and/or NPDES studies dated _________________ (*roadway designer's initials and date*)."

The roadway designer will forward the revised sheets to the Preconstruction Support Operations Office, which will be forwarded to the Rights of Way Office and Environmental Services Office. If hydraulic/NPDES revisions are made to parcels that have already been acquired (including permission granted), then the hydraulic designer and the roadway designer should attempt a resolution before finalizing the revisions and revisiting the property owner.

12.1.6 Construction Limits

The determination and delineation of accurate construction limits on the plans are critical to the establishment of the new right-of-way boundaries. After completing the horizontal and vertical design on the plans and plotting the templates on cross sections, the designer is responsible for showing the construction limits on the plans. Measure the construction limits at each cross section location and show the construction line with connecting dashed lines in the plan view. Carefully review areas not covered by cross sections to ensure that the limits of construction represent the actual conditions expected during construction. Label the points on the limit lines with a distance from the centerline and use a "C" or "F" to denote cut or fill. Ensure construction limits are also shown adjacent to interchange ramps, loops, flyovers, cross roads and parallel drainage courses. In special cases, it may be desirable to show dual construction limits for clarification purposes.

12.1.7 Breaks in Right of Way

Where it is necessary to have breaks in the right of way and the location is near a property line, the designer should not use the property line as a break; however, when it is necessary to convert permission to right of way, it is acceptable to tie right-of-way breaks to property lines. For more information on how to show breaks in the Right of Way Plans, see Chapter 22 "Plan Sheets Preparation."

12.1.8 Triangular Areas

Triangular areas are necessary for construction purposes, as well as sight distance control, and should be acquired as normal right of way. Sight distance, for vehicles approaching at-grade intersections, is a major design consideration for most roadway projects. Generally, a driver's line of sight will fall outside the limits of uniform right of way at the intersection of two roadways.

All quadrants are to be checked for sight distance requirements to ensure sufficient line of sight area is available.

Within triangular areas at intersections, be aware of proper shoulder width, ditch construction around the radius, traffic control devices and placement of pipe or structures that may require extra right of way.

Once the required triangular area is determined, clearly show this area on the Right of Way Plans.

12.1.9 Right of Way on Sharp Horizontal Curves

Horizontal sight restrictions may be caused by retaining walls, bridge parapet walls, cut slopes, trees, buildings, etc., on the inside of curves. If the obstruction is outside of the right of way, consideration of obtaining the additional right of way may be merited on new construction projects. See Section 5.4 for guidance on measuring the horizontal sight line offset.

12.1.10 Traffic Control Devices

Additional right of way may be required for the construction and maintenance requirements for traffic control devices. Early coordination with the traffic designer is required to determine right-of-way needs. The road designer is required to review all traffic control improvements required for the project and ensure adequate right of way is available. Of critical importance are the locations, dimensions and construction slope requirements for all underground footings, as well as, traffic signal poles, and all supporting appurtenances.

12.1.11 Outfall/Infall Ditches

Outfall/infall ditches are defined as ditches that intercept normal roadway longitudinal ditches and/or culvert ends and carry surface water from and to the roadway facility. They may be in the form of existing ditches or proposed ditches.

Outfall ditches require right of way or permission be obtained from the property owner to construct or clean. In general, right of way should be acquired for outfall ditches on all Federal-aid projects, and on other projects, where ditches are determined to be necessary for the overall function of the drainage system and protection of the roadway. New right of way should be of sufficient width to provide for construction activities and future maintenance. Unless circumstance dictates otherwise, a total width of 30 feet, 10 feet on one side and 20 feet on the other, is generally considered to be acceptable. Where a specific length outfall is requested, measure it from the centerline of the intersecting road.

12.1.12 Channel Changes

Channel changes are defined as relocations of streambeds. Generally, channel changes are not preferred due to environmental concerns. Where necessary, relocations may be situated parallel to, or crossing, the project. The new right-of-way limits should be sufficient to cover construction and future maintenance requirements. All relocation of streambeds require coordination with the United States Army Corps of Engineers; coordinate with SCDOT's Environmental Services Office.

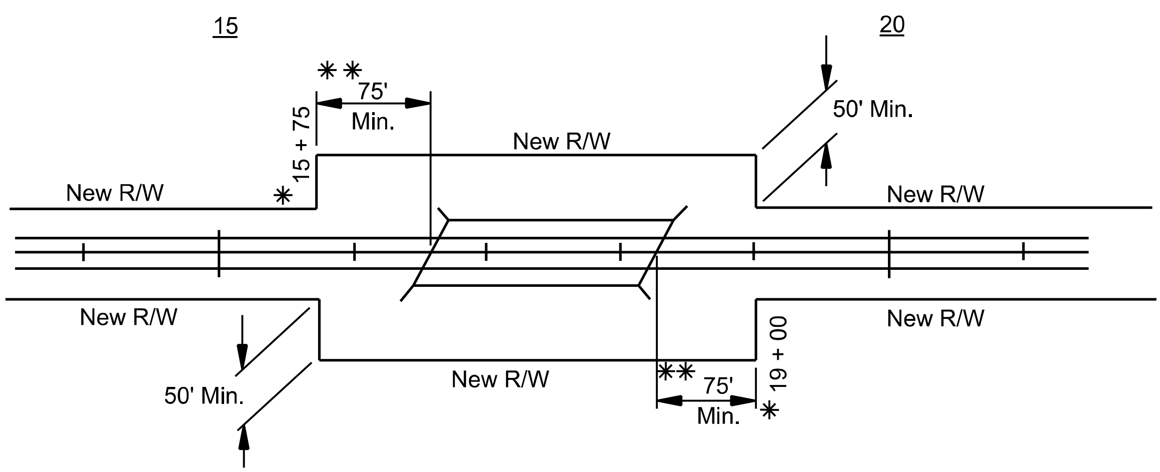
12.1.13 Culvert Sites

Provide a minimum space of 10 feet between the wing wall ends of a box culvert and the present or new right of way. Additional right of way may be required to encompass permanent erosion control devices (e.g., energy dissipaters, paved liners, scour protection devices) placed at the ends of culverts.

12.1.14 Bridge Sites

In general, provide a minimum right-of-way width of 75 feet each side of the structure centerline, to a point 75 feet from each end of the bridge, on all projects having a single two-lane bridge. Where multilane or divided highway structures are proposed, the distance between the existing or proposed roadway approach right of way and the additional right of way required for bridges should be established for the specific site conditions. In addition, consider construction staging, access for construction and maintenance when establishing permanent right of way and temporary access for construction. See Figure 12.1-D for an illustration of the minimum right of way required at bridges.

Carefully review the point where the extra bridge right of way turns 90 degrees and intersects the approach roadway right of way. When additional embankment is placed for guardrail, the slopes can cut off access around the bridge ends for maintenance. A transitioned right of way may be appropriate in these situations.



✳ Should be to nearest even station or 25' interval beyond 75'.

✳ ✳ Adjust the length as required to preclude cutting off access to underside of bridge by the toe of fill or installation of guardrail.

RIGHT OF WAY AT BRIDGE SITES
Figure 12.1-D

12.1.15 Bridge Construction Access

During the construction of bridges, the contractor's equipment has to be positioned near the new bridge site to facilitate construction activities. This location will be at one of the four corners of the new bridge and will be bounded by the body of water, railroad or highway being crossed, the right-of-way line and a distance of a transverse line 75 feet parallel to the construction centerline from the body of water, railroad or highway. In order to provide access to this location for large equipment (e.g., a crane), an access road a short distance along the right-of-way line may have to be made available to the contractor. The access road and equipment set-up site will be noted as the "Bridge Construction Access (BCA)" (see Figure 12.1-E) and will be shown on the plans. During the Design Field Review, the District representative will provide the location of the BCA. The designer will sketch the location on the plans during the field review. The Right of Way Plans will show this access by a unique line that can be found in the custom line style palette and is shown in Figure 12.1-E.

The area within the BCA line will be cleared and grubbed during construction. A silt fence will be installed along the outer most limits of the BCA. Permission should be obtained when the BCA is shown outside the right of way, but may have to be encompassed with permanent right of way due to the amount of work required within its boundaries. A minimum of 20 feet from the fill slope is required to the BCA line. The BCA line will only be shown at one corner of the future bridge site, unless conditions require additional access on other corners of the bridge. The BCA is not authorized in wetland areas unless otherwise authorized by issuance of a United States Army Corps of Engineers permit; coordinate with SCDOT Environmental Services Office.

--- BCA --- BCA --- BCA ---

BCA LINE
Figure 12.1-E

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12.2 ELEMENTS OF RIGHT OF WAY

12.2.1 Property Information

In the early stages of the plan development process, each parcel of property where acquired right of way is anticipated will be given a consecutive tract number assigned in the direction of stationing. While property ownership may change during development of the project, the pre-assigned parcel identification numbers will not change.

Should realignments affect additional properties, they may be assigned new whole numbers, continuing consecutively from the last whole number used. Should a pre-numbered property be divided or sub-divided, identify the new parts of the original tract using the original whole number followed by a letter suffix (e.g., 12A, 12B, 12C). Should a previously numbered parcel be deleted, the notation "OMITTED" may be entered under the column of the summary sheet showing the property owner's name.

Chapter 22 "Plan Sheets Development" provides guidance on the property information that is required on the plan sheets.

Where a parcel of land is severed by the proposed improvement, areas remaining left and right of the mainline facility are indicated on the Right of Way Plans via a summation of the total areas.

12.2.2 Dedicated Right of Way

Dedicated right of way is property conveyed to a public-governmental entity by a private source for public use and benefit. This property must be accepted by a public body for the dedication to be valid.

12.2.3 Property Closures

Property closures, developed by survey parties, generally depict the metes and bounds description of property as obtained from available deed and plat records. Because the property work files are preserved, it is not mandatory that the metes and bounds information be repeated on the property closures depicted in the Right of Way Plans. Each property closure shown in the plans must clearly depict the graphical boundaries of the entire parcel and the graphical area to be acquired. Where right of way is necessary, the affected tracts are numbered, computed and recorded under the property owners name on the Right of Way Data Sheet.

Generally, on bridge replacement projects, it is not necessary to close the property. Property lines and owner names should be shown as on a secondary project.

All projects, except secondary projects, non-surveyed projects and bridge projects, are required to have all affected properties numbered and closed.

When condemnation is required on non-surveyed and bridge projects, request a survey for the affected properties as soon as practical.

12.2.4 Encroachments

Highway encroachments occur when persons other than the Department's staff or the Department's authorized agents place items within existing highway right of way. No encroachments of any type are allowed on highway right of way without an official encroachment permit.

12.2.5 Land Locked Parcels

A parcel of land is land locked when it has no legal access. One form of legal access is provided via public roads. It may also be provided via rights of ingress/egress through abutting properties through some form of legal instrument between two or more parties (e.g., a condition of the deed, official agreement easement).

A parcel of land may be land locked as it exists or may become land locked as a result of highway improvements. In either case, note land locked properties in the comments column on the Right of Way Acquisition Summary Sheet and on the Plan and Profile sheets. In some cases, the land locked parcel may be incorporated into the right of way or may be acquired as excess land. The Rights of Way Office will make the final determination whether to acquire these properties.

12.2.6 Permissions

Permissions allow the Department and/or its contractors the right to enter the property to perform specific actions described in the Permission. Permissions should only be obtained for temporary construction activities and should not be used for long-term maintenance. Be aware that permissions are temporary and can be revoked at any time by the property owner. Permissions do not transfer with the sale of property. A new permission must be secured from the new property owner.

12.2.7 Right-of-Way Deeds/Easements

The primary function of Right of Way Plans and instruments is to provide the necessary tools by which the Department acquires all real property in the form of fee simple titles or easements for the State's secondary, primary and Interstate roads systems.

Standard forms of right-of-way instruments include titles to real estate, right-of-way easements, permission forms and notices of condemnation. These standard documents are available from the Rights of Way Office.

12.2.7.1 Deed Descriptions

The description portion of a right-of-way deed defines the parcel to be acquired and should reference the survey centerline using the station and offset method, or to an exhibit developed from the Plans. A reference to the Right of Way Plans project ID and tract number contained in each right-of-way deed.

12.2.7.2 Permanent Easements (Outside the Right of Way)

Permanent easements provide the Department the right to occupy the property of others for a designated purpose, usually construction and maintenance of a highway or drainage rights, in perpetuity. The easement can only be extinguished by the Department, or by court order.

12.2.7.3 Temporary Construction Easements

Temporary easements provide the Department the right to occupy the property of others for a designated purpose (e.g., drainage, demolition access, detour construction) over a limited period of time. The purpose and time period must be clearly defined. The time may be designated as actual completion of the work. At the end of the period, the easement is extinguished. The note on the plans should state, "OBTAIN TEMPORARY RIGHT OF WAY."

12.2.7.4 Property Records

The permanent records of all real property owned by the Department are in custody of the Rights of Way Office. All deeds for property acquired prior to April 1, 1988, are recorded in the vault of the Department's Rights of Way Office in headquarters. Title to all Department-owned property, acquired after April 1, 1988, is recorded in the Register Mesne Conveyance/*Register of Deeds* in the county courthouse in which the property is situated, and filed in the vault of the Department's Rights of Way Office headquarters. Deed and title are considered Instruments of Conveyances.

12.2.8 <u>Special Features</u>

The items in the following sections will be occasionally encountered on highway projects.

12.2.8.1 Retaining Walls

Where it becomes necessary to construct retaining walls to contain cut or fill slopes, the backs of walls, the face of a wall farthest from the traveled way, is established as the right-of-way limit. Depending upon the design of the wall, right of way may be required to construct and maintain the wall. In situations where the landowner negotiates for a wall, in lieu of slope permission, the right of way may be established as the front of the wall; the wall will become the property of the landowner. All future maintenance will be the responsibility of the landowner.

12.2.8.2 Cemeteries

Cemeteries should be identified at the beginning of preliminary studies and referenced in the Project Screening Tool (PST). Avoid disturbances to cemeteries to the maximum extent possible. If no alternative appears available, contact the Director of Rights of Way immediately. Locate all graves near the improvement via surveys and show them on the plans.

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12.3 REFERENCES

1. *Right of Way Manual,* South Carolina Department of Transportation, current edition.

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Chapter 13

RESERVED

SOUTH CAROLINA HIGHWAY DESIGN MANUAL

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Chapter 14

LOCAL ROADS AND STREETS

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 14
LOCAL ROADS AND STREETS

Local roads and streets primarily serve as access roads to farms, residences, businesses and other abutting properties. They distribute traffic to highways in the higher functional classification network.

This chapter discusses the criteria used in the design of local roads and streets. Information that is also applicable to the design of local rural roads and local urban streets is included in the following chapters:

- Chapters 3 “Basic Design Controls,” Chapter 4 “Sight Distance,” Chapter 5 “Horizontal Alignment,” Chapter 6 “Vertical Alignment” and Chapter 7 “Cross Section Elements” provide guidance on the geometric design elements.
- Chapter 9 “Intersections” provides information on the design of intersections, including intersection alignment, left- and right-turn lanes and curb radii.

14.1 GENERAL

14.1.1 Descriptions

1. Rural. A major part of the rural highway system consists of two-lane local roads. These roadways should be designed to accommodate the highest practical criteria compatible with traffic and topography.
2. Urban. A local urban street is a public roadway for vehicular travel including public transit and refers to and includes the entire area within the right of way. The street also serves pedestrian and bicycle traffic and usually accommodates public utility facilities within the right of way. The development or improvement of these streets should be based on a functional street classification that is part of a comprehensive community development plan. The design criteria should be appropriate for the planned development. The two major design controls are (1) the type and extent of urban development with its limitations on right of way, and (2) zoning or regulatory restrictions. Local streets primarily serve to provide access to adjacent residential development areas. The overriding consideration is to foster a safe and pleasant environment whereas the convenience of the motorist is secondary. Other local streets not only provide access to adjacent development, but also serve limited through traffic. Traffic service features may be an important concern on these streets (e.g., traffic signals, left-turn lanes).

14.1.2 State Highway Local Roads and Streets

Local roads and streets are divided into the following categories:

1. Group 1. Group 1 roads and streets are typically located in subdivisions or residential areas.

2. Group 2. A Group 2 road or street:

- is ½ mile or less in length;
- is not a major connector (e.g., to a major traffic generator);
- does not dead-end; and
- has an AADT of less than 250.

3. Group 3. A Group 3 road or street:

- is between ½ and 1 mile in length,
- is not a connecting road or street, and
- has an AADT of 500 or less.

4. Group 4. Group 4 roads and streets are all other local rural roads and local urban streets, except subdivision streets.

The geometric design criteria presented in Section 14.3 for Groups 1 through 4 is the minimum criteria for State local rural road and local urban street projects.

14.2 DESIGN ELEMENTS

The design criteria discussed in this section applies to all local roads and streets included in the State Highway System. For roads and streets maintained by local governments, additional guidance can be found in the AASHTO publications *A Policy on Geometric Design of Highways and Streets* and *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)*.

14.2.1 Traffic Volume

Traffic volume is not usually a major factor in determining the geometric design criteria to be used in designing residential streets. Traditionally, these streets are designed with a standard two-lane cross section, but a four-lane cross section may be appropriate in certain urban areas, as governed by traffic volume, administrative policy or other community considerations (e.g., pedestrians, bicyclists). However, to provide the requisite traffic mobility and safety together with the essential economy in construction, maintenance and operation, these roads and streets must be planned, located and designed to be suitable for predictable traffic operations for all modes of travel and must be consistent with the development and culture abutting the right of way.

For streets serving industrial or commercial areas, traffic volume may be a major factor. For these streets, the AADT projected to 20 years is desirable.

14.2.2 Design Speed

The design speed establishes the range of design values for many of the geometric elements of the highway (e.g., sight distance, horizontal alignment, vertical alignment). The selected design speed should be high enough so that an appropriate regulatory speed limit will be less than or equal to it. Desirably, the speed at which drivers are operating comfortably will be close to the posted speed limit. See Section 3.5.2 and the FHWA publication *Mitigation Strategies for Design Exceptions* for additional guidance on the selection of design speeds.

Design speeds for rural local roads are based on terrain, traffic volumes, driver expectancy and alignment, and may range from 20 to 60 miles per hour. Urban design speeds for local streets can range from 20 to 30 miles per hour, depending on available right of way, terrain, adjacent development, likely pedestrian presence and other site controls. Lower speeds apply in CBD and in more developed areas, while higher speeds are more applicable to outlying suburban and developing areas.

The geometric design tables in Section 14.3 provide the applicable design speeds for local roads and streets.

14.2.3 Sight Distances

See Chapter 4 “Sight Distance” for guidance on stopping, decision, passing and intersection sight distances. Section 14.3 provides specific sight distance values for local roads and streets.

14.2.4 Alignment

The horizontal and vertical alignment should complement each other and should be considered in combination to achieve appropriate safety, capacity and appearance for the type of improvement proposed. Proper combinations of curvature, tangents, grades, variable median widths and separate roadway elevations all combine to enhance safety and aesthetics of local roads and streets. When designing the horizontal and vertical alignments, the designer should provide the most favorable alignment practical consistent with the environmental impact, topography, terrain, design traffic volume and reasonably obtainable right of way. Consider the following:

1. Low-Speed Urban Streets. Where superelevation is required on low-speed urban streets ($V_d \leq 45$ mph), use AASHTO Method 2 to determine the design superelevation. See Section 5.3.3 for minimum radii and superelevation rates for low-speed urban streets.

2. Horizontal and Vertical Combinations. Consider the relationship between horizontal and vertical alignments simultaneously to obtain a desirable condition. Section 6.2.2 discusses this relationship in detail and its effect on aesthetics and safety.

3. Minimum Grades. Desirably, the longitudinal grade should be a minimum of 0.5 percent. For curbed facilities and bridges, it is necessary to provide a minimum longitudinal grade of 0.3 percent to facilitate drainage. For curbed sections, ensure curb profiles provide positive drainage. For uncurbed facilities, a minimum longitudinal grade of 0.0 percent may be considered if adequate cross slopes are provided. Ensure superelevation transitions are not developed in areas with 0 percent grade. Special ditch grades may be necessary to ensure proper drainage.

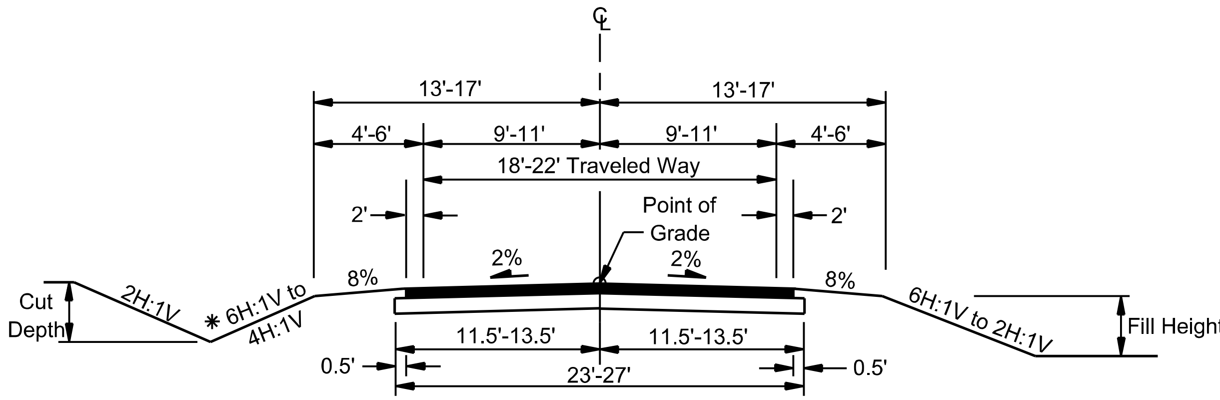
14.2.4.1 Typical Sections

The following figures present typical sections for local roads and streets:

- Figure 14.2-A – Typical Local Rural Road or Local Urban Street with Shoulders
- Figure 14.2-B – Typical Local Rural Road or Local Urban Street with Valley Gutters

14.2.4.2 Number of Lanes

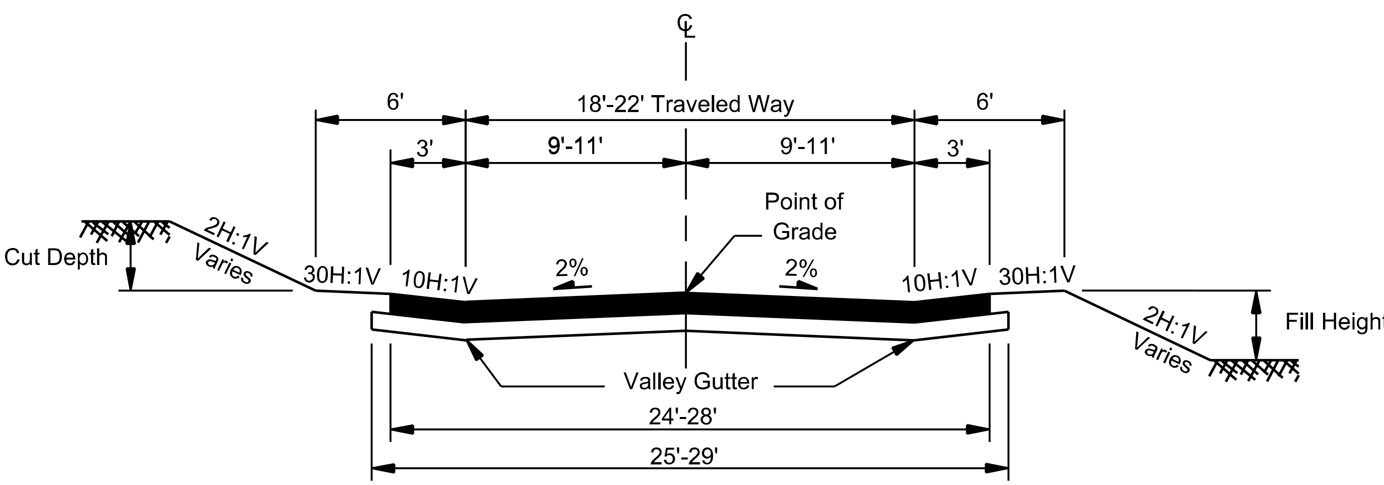
Two lanes usually accommodate rural local roads. A large majority of urban residential streets provide two travel lanes with or without parking lanes on one or both sides.



*This slope may vary between a minimum slope of 6H:1V to a maximum slope of 4H:1V. Where a deeper ditch than provided by a 4H:1V slope is necessary for drainage purposes, continue the 4H:1V slope until the necessary depth has been obtained. This will place the ditch further from the roadway. Provide a separate profile for special ditch grades.

Note: See Section 14.3 for specific road group criteria.

TYPICAL LOCAL RURAL ROAD OR LOCAL URBAN STREET
(With Shoulders)
Figure 14.2-A



Note: See Section 14.3 for specific road group criteria.

TYPICAL LOCAL RURAL ROAD OR LOCAL URBAN STREET
(With Valley Gutter)
Figure 14.2-B

14.2.4.3 Travel Lane and Shoulder Widths

Travel lane widths may be 9 feet to 11 feet depending upon the road group type.

For rural roads, provide a minimum 4-foot shoulder or a 6-foot shoulder for Group 4 roads. The shoulder width includes a minimum paved width of 2 feet. Where bicycles are to be accommodated on the shoulder, the designer should provide a minimum paved shoulder width of 4 feet.

In constrained urban areas with low speeds, the shoulder width may be just the 3-foot valley gutter width. The use of curb and gutter and valley gutter sections are common on urban streets to reduce right-of-way requirements.

For specific lane and shoulder width criteria for local roads and streets, see the geometric design tables in Section 14.3.

14.2.4.4 Cross Slopes

Use a cross slope of 2.00 percent for up to two lanes plus one half the width of the flush median or TWLTL. Crown the pavement at the center of the TWLTL and use a cross slope of 2.00 percent away from the centerline for all lanes on three- and five-lane highways. If a roadway profile grade is less than 2.00 percent, the designer may consider using a cross slope of 2.50 percent for the outside lane to improve drainage. See Section 7.2.3.3.

For paved shoulders greater than 4 feet, provide a shoulder cross slope of 4.00 percent. For paved shoulders less than or equal to 4 feet, the cross slope should match the adjacent travel lane slope. For earth shoulders, provide a shoulder cross slope of 8.00 percent.

For bridge cross slopes, see the *SCDOT Bridge Design Manual*.

14.2.4.5 Auxiliary Lanes

Auxiliary lanes (e.g., passing lanes, parking lanes, turn lanes) are lanes beyond the through travel lanes intended for use by vehicular traffic for specific functions. Desirably, auxiliary lanes will have the same width and cross slope as the adjacent through lanes, although in many cases a lesser width may be appropriate. The geometric design tables in Section 14.3 present lane and shoulder widths for auxiliary lanes.

14.2.4.6 Bicycle Accommodations

For accommodation of bicyclists, the designer should review the guidance provided in Section 11.11.

14.2.4.7 Medians

Medians are generally not provided on local roads and streets. If a median is considered on an urban street, they may be one of the following median types:

1. Flush Medians. Flush medians provide an area for left-turn movements and permit direct access to adjoining properties. This allows for numerous unrestricted conflict points. The flush median may serve as refuge for disabled vehicles and as a temporary lane for emergency vehicles. The two-way, left-turn lane (TWLTL) is considered a flush median. Desirably, the roadway cross section with a flush median will allow development of a TWLTL, if applicable.

2. Raised Medians. Raised medians restrict left-turn movements to select locations, which allows for better access management. This median may provide a refuge area for pedestrians and an open space for aesthetic considerations.

For guidance on medians and TWLTL, see Chapter 7 “Cross Section Elements.”

14.2.4.8 Right of Way

Providing right-of-way widths that accommodate construction, drainage and proper maintenance of a highway is an important part of the overall design. Wider right of way allows for gentler side slopes, which results in reduced crash severity potential and easier maintenance operations. Right of way is typically configured to accommodate all proposed cross section elements throughout the project (e.g., travel lanes, shoulders, medians, parking lanes, bike lanes, sidewalks, ditches, outer slopes). In developed areas, it may be necessary to limit the right-of-way width. A uniform right-of-way width is preferred; however, do not base the width on the critical point of the project. A critical point may occur where the side slopes extend beyond the normal right of way, for clear areas at the bottom of traversable slopes, for wider clear areas on the outside of curves, where greater sight distance is desirable, at intersections and junctions with other roads, at railroad-roadway grade crossings, for environmental considerations and for maintenance access.

14.2.5 Roadside Safety

The designer should provide adequate horizontal clearance between the traveled way and roadside obstructions on local roads and streets. The designer should provide roadside clear zones as discussed in the AASHTO *Roadside Design Guide*.

14.2.6 Bridges

In general, bridge widths should match the approach roadway width (traveled way plus shoulders). See Section 7.5.1.1 for additional information on bridge widths.

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14.3 TABLES OF DESIGN CRITERIA

The geometric design tables in this section present the Department's design and alignment criteria for local rural roads and local urban streets. The designer should consider the following when using these figures:

1. Functional Classification. To determine the latest functional classification of a facility, the designer should contact Road Data Services.

2. Applicability. Note that some of the cross-section elements included in the figures (e.g., TWLTL) are not automatically warranted in the project design. The values in the figures only apply after the decision has been made to include the design element in the highway cross section.

3. Manual Section References. These figures are intended to provide a concise listing of design values for easy use. However, the designer should review the manual section references for more information on the design elements.

4. Footnotes. The figures include many footnotes, which are identified by a number in parentheses (e.g., **(3)**). The information in the footnotes is critical to the proper use of the design tables.

The following design tables are provided for local rural roads and urban local streets:

- Figure 14.3-A — "Geometric Design Criteria for Local Rural Roads (New Construction/ Reconstruction)"
- Figure 14.3-B — "Alignment Criteria for Local Rural Roads (New Construction/ Reconstruction)"
- Figure 14.3-C — "Geometric Design Criteria for Local Urban Streets (New Construction/ Reconstruction)"
- Figure 14.3-D — "Alignment Criteria for Local Urban Streets (New Construction/ Reconstruction)"

Design Element				Manual Section	Design Criteria			
					Group 1	Group 2	Group 3	Group 4
Design Controls	Design Forecast Year **(1)**			14.2.1	20 Years	20 years	20 years	20 years
	Minimum Design Speed			14.2.2	**(2)**	20 mph	30 mph	35 mph
	Access Control			3.8	Controlled by Regulation			
	Level of Service			3.6.4	N/A	N/A	N/A	N/A
Cross Section Elements	Travel Lane Width			14.2.4	Min.: 9 ft	Des.: 10 ft Min.: 9 ft	Des.: 11 ft, Min.: 10 ft	11 ft
	Shoulder Width **(3)**	Total		14.2.4	4 ft	Des.: 6 ft	Min.: 4 ft	6 ft
		Paved			2 ft	2 ft	2 ft	2 ft
	Auxiliary Lanes	Lane Width		14.2.4	N/A	N/A	N/A	Min. 11 ft **(4)**
		Shoulder Width	Total		N/A	N/A	N/A	6 ft
			Paved		N/A	N/A	N/A	2 ft
	Cross Slope	Travel Lane		14.2.4	2.00%			
		Auxiliary Lane		14.2.4	N/A	N/A	N/A	2.00%
		Shoulder	Paved **(5)**	14.2.4	2.00%			
			Unpaved		8.00%			
	Bicycle	Bike Lane Width **(6)**		11.11	4 ft			
		Shared Roadway Width			N/A	N/A	N/A	14 ft Outside TL
	Sidewalk Width			7.3.3	5 ft			
	Median	Width (TWLTL)		7.4	N/A	N/A	N/A	15 ft
		Flush/TWLTL Slopes			N/A	N/A	N/A	2.00%
	Right-of-Way Width			14.2.4	Project Specific			
Roadway Elements	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V to 4H:1V			
			Ditch Type		V-Ditch			
			Back Slope		2H:1V			
		Fill Section	0 ft – 5 ft		6H:1V			
			5 ft – 10 ft		4H:1V			
			> 10 ft		2H:1V			
	Clear Zone				**(7)**			

GEOMETRIC DESIGN CRITERIA FOR LOCAL RURAL ROADS
(New Construction/Reconstruction)
Figure 14.3-A
(Continued on next page)

<table>
<tr><th colspan="3">Design Element</th><th>Manual Section</th><th colspan="2">Rural</th></tr>
<tr><td rowspan="15">Structures</td><td rowspan="2">New Bridges</td><td>Structural Capacity</td><td>7.5.1</td><td colspan="2">HL-93</td></tr>
<tr><td>Clear Roadway Width</td><td>14.2.6</td><td colspan="2">(8)</td></tr>
<tr><td rowspan="2">Existing Bridges to Remain in Place</td><td>Structural Capacity</td><td rowspan="2">14.2.6</td><td colspan="2">(9)</td></tr>
<tr><td>Clear Roadway Width</td><td colspan="2">(8)</td></tr>
<tr><td rowspan="5">Vertical Clearance (Local Road Under) (10a)</td><td>New and Replaced Overpassing Bridges (10b)</td><td rowspan="5">6.6</td><td colspan="2">16 ft – 0 in</td></tr>
<tr><td>Existing Overpassing Bridges</td><td colspan="2">14 ft – 0 in</td></tr>
<tr><td>Pedestrian Bridges</td><td colspan="2">18 ft – 0 in</td></tr>
<tr><td>Overhead Signs</td><td colspan="2">17 ft – 6 in</td></tr>
<tr><td>Overhead Utilities</td><td colspan="2">Coordinate with Utilities Office</td></tr>
<tr><td rowspan="2">Vertical Clearance (Local Road Over)</td><td>Railroads</td><td>6.6</td><td colspan="2">23 ft – 0 in</td></tr>
<tr><td>Underpass Width</td><td>7.5.2</td><td>Approach Roadway Width Including Sidewalks, where applicable</td><td>Traveled Way plus Clear Zone</td></tr>
<tr><td rowspan="4">Vertical Clearance (Over Water)</td><td>Navigable Water</td><td rowspan="4">6.6</td><td colspan="2">Coordinate with Environmental Services Office</td></tr>
<tr><td>Major Lakes & Reservoirs (with boat traffic)</td><td colspan="2">8 ft – 0 in above the high water mark</td></tr>
<tr><td>Rivers</td><td colspan="2">2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.</td></tr>
<tr><td>Tidal Waters</td><td colspan="2">2 ft above the 10-year high water elevation including wave height.</td></tr>
</table>

GEOMETRIC DESIGN CRITERIA FOR LOCAL RURAL ROADS
(New Construction/Reconstruction)
Figure 14.3-A
(Continued on next page)

Footnotes for Figure 14.3-A

(1) Design Forecast Year. Table values are desirable. For rural roads, the design year may be current traffic volumes.

(2) Minimum Design Speed. Design speed is not a major factor for Group 1 roads and streets. Select a design speed based on available right of way, terrain, likely pedestrian presence, adjacent development and other area controls.

(3) Shoulder Width. Shoulders should be increased by 3.5 feet where guardrail is used.

(4) Auxiliary Lane Width. The auxiliary lane width should be the same as the adjacent travel lane.

(5) Shoulder Cross Slope. For paved shoulders wider than 4 feet, use a 4.00 percent shoulder cross slope.

(6) Bicycle Facilities Lane Width. If curb and gutter is provided, provide a 4-foot width from the face of curb. For design speeds greater than 45 miles per hour, increase the bike lane width in accordance with AASHTO *Guide for the Development of Bicycle Facilities*.

(7) Clear Zones. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(8) Bridge Widths. Bridge width is equal to the width of roadway section (outside shoulder to outside shoulder). See Section 7.5.1.1 for bridge widths.

(9) Structural Capacity (Existing Bridges). Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(10) Vertical Clearance (Local Roads Under).

a. Provide the vertical clearance over the entire traveled way, shoulders and any anticipated future widening.

b. Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR LOCAL RURAL ROADS
(New Construction/Reconstruction)
Figure 14.3-A
(Continued)

Design Element		Manual Section	Design Speed								
			20 mph	**25 mph**	**30 mph**	**35 mph**	**40 mph**	**45 mph**	**50 mph**	**55 mph**	**60 mph**
Stopping Sight Distance **(1)**		4.1	115 ft	155 ft	200 ft	250 ft	305 ft	360 ft	425 ft	495 ft	570 ft
Passing Sight Distance		4.2	400 ft	450 ft	500 ft	550 ft	600 ft	700 ft	800 ft	900 ft	1000
Decision Sight Distance **(2)**		4.3	-	-	220 ft	275 ft	330 ft	395 ft	465 ft	535 ft	610 ft
Intersection Sight Distance **(3)**		4.4	225 ft	280 ft	335 ft	390 ft	445 ft	500 ft	555 ft	610 ft	665 ft
Minimum Radii	e_{max} = 8%	5.2							758 ft	960 ft	1200
	e_{max} = 6%		81 ft	144 ft	231 ft	340 ft	485 ft	643 ft	833 ft		
Superelevation Rate **(4)**		5.3	6%	6%	6%	6%	6%	6%	6 / 8%	8%	8%
Horizontal Sight Line Offset **(5)**		5.4	20 ft	20 ft	21 ft	23 ft	24 ft	25 ft	27 ft 30 ft	32 ft	34 ft
Min. Vertical (K-values) **(6)**	Crest	6.5	7	12	19	29	44	61	84	114	151
	Sag		17	26	37	49	64	79	96	115	136
Maximum Grade	Level	6.3.1	8%	7%	7%	7%	7%	6%	6%	6%	5%
	Rolling		11%	11%	10%	10%	9%	8%	7%	7%	6%
	Mountain		16%	15%	14%	13%	12%	10%	10%	10%	n/a
Minimum Grade **(7)**		6.3.2	Des.: 0.5% Min.: 0.0%								

Footnotes

(1) Stopping Sight Distance. Table values are for passenger cars on level grade.

(2) Decision Sight Distance. Table values are for stop on a rural road, Avoidance Maneuver A. See Section 4.3 for other maneuvers.

(3) Intersection Sight Distance. Table values are for passenger cars for assumed conditions described in Figure 4.4-C. See Section 4.4 for other conditions.

(4) Superelevation Rate. See Section 5.3 for superelevation rates based on e_{max}, design speed and radii of horizontal curves.

(5) Horizontal Sight Line Offset. Table values provide the necessary middle ordinate assuming the design speed, stopping sight distance and minimum radii based on an e_{max} = 6 percent for design speeds 20 to 50 miles per hour and e_{max} = 8 percent for design speeds of 50 miles per hour or greater.

(6) Vertical Curvature (K-Value). K-values are based on the level stopping sight distances.

(7) Minimum Grade. The minimum grade of 0.0 percent can only be used on ditch sections where there is an adequate roadway cross slope and ditch grade. Ensure superelevation transitions are not developed in areas with 0.0 percent grade. Special ditch grades may be necessary to ensure proper project runoff management.

ALIGNMENT CRITERIA FOR LOCAL RURAL ROADS
(New Construction/Reconstruction)
Figure 14.3-B

Design Element				Manual Section	Design Criteria Group 1	Group 2	Group 3	Group 4
Design Controls	Design Forecast Year **(1)**			14.2.1	20 Years	20 years	20 years	20 years
	Minimum Design Speed			14.2.2	**(2)**	20 mph	20 mph	30 mph
	Access Control			3.8	Controlled by Regulation			
	Level of Service			3.6.4	N/A	N/A	N/A	N/A
Cross Section Elements	Travel Lane Width			14.2.4	Min.: 9 ft	Des.: 10 ft Min.: 9 ft	Des.: 11 ft, Min.: 10 ft	Min.: 11 ft
	Shoulder Width **(3)**	Total		14.2.4	4 ft or C/G	Des.: 6 ft Min.: 4 ft or C/G		6 ft or C/G
		Paved			2 ft or C/G	2 ft or C/G	2 ft or C/G	2 ft or C/G
	Auxiliary Lanes	Lane Width		14.2.4	N/A	N/A	N/A	Min. 11 ft **(4)**
		Shoulder Width	Total		N/A	N/A	N/A	6 ft or C/G
			Paved		N/A	N/A	N/A	2 ft or C/G
	Parking Lane Width			7.2.7	8 ft – 10 ft			
	Cross Slope	Travel Lane		14.2.4	2.00%			
		Auxiliary Lane		14.2.4	N/A	N/A	N/A	2.00%
		Shoulder	Paved **(5)**	14.2.4	2.00%			
			Unpaved		8.00%			
	Bicycle	Bike Lane Width **(6)**		11.11	4 ft			
		Shared Roadway Width			N/A	N/A	N/A	14 ft Outside TL
	Curb & Gutter	Type **(7)**		7.2.8	Vertical, Sloping or Valley Gutter			
		Width			2 ft			
	Sidewalk Width			7.3.3	5 ft			
	Median	Width (TWLTL)		7.4	N/A	N/A	N/A	15 ft
		Flush/TWLTL Slopes			N/A	N/A	N/A	2.00%
	Right of Way Width			14.2.4	Project Specific			
Roadway Elements	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V to 4H:1V			
			Ditch Type		V-Ditch			
			Back Slope		2H:1V			
		Fill Section	0 ft – 5 ft		6H:1V			
			5 ft – 10 ft		4H:1V			
			> 10 ft		2H:1V			
	Clear Zone				**(8)**			

GEOMETRIC DESIGN CRITERIA FOR LOCAL URBAN STREETS
(New Construction/Reconstruction)
Figure 14.3-C
(Continued on next page)

<table>
<tr><th colspan="3">Design Element</th><th>Manual Section</th><th colspan="2">Rural</th></tr>
<tr><td rowspan="19">Structures</td><td rowspan="2">New Bridges</td><td>Structural Capacity</td><td>7.5.1</td><td colspan="2">HL-93</td></tr>
<tr><td>Clear Roadway Width</td><td>14.2.6</td><td colspan="2">(9)</td></tr>
<tr><td rowspan="2">Existing Bridges to Remain in Place</td><td>Structural Capacity</td><td rowspan="2">14.2.6</td><td colspan="2">(10)</td></tr>
<tr><td>Clear Roadway Width</td><td colspan="2">(9)</td></tr>
<tr><td rowspan="5">Vertical Clearance (Local Road Under) (11a)</td><td>New and Replaced Overpassing Bridges (11b)</td><td rowspan="5">6.6</td><td colspan="2">16 ft – 0 in</td></tr>
<tr><td>Existing Overpassing Bridges</td><td colspan="2">14 ft – 0 in</td></tr>
<tr><td>Pedestrian Bridges</td><td colspan="2">18 ft – 0 in</td></tr>
<tr><td>Overhead Signs</td><td colspan="2">17 ft – 6 in</td></tr>
<tr><td>Overhead Utilities</td><td colspan="2">Coordinate with Utilities Office</td></tr>
<tr><td rowspan="2">Vertical Clearance (Local Road Over)</td><td>Railroads</td><td>6.6</td><td colspan="2">23 ft – 0 in</td></tr>
<tr><td>Underpass Width</td><td>7.5.2</td><td>Approach Roadway Width Including Sidewalks, where applicable</td><td>Traveled Way plus Clear Zone</td></tr>
<tr><td rowspan="4">Vertical Clearance (Over Water)</td><td>Navigable Water</td><td rowspan="4">6.6</td><td colspan="2">Coordinate with Environmental Services Office</td></tr>
<tr><td>Major Lakes & Reservoirs (with boat traffic)</td><td colspan="2">8 ft – 0 in above the high water mark</td></tr>
<tr><td>Rivers</td><td colspan="2">2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.</td></tr>
<tr><td>Tidal Waters</td><td colspan="2">2 ft above the 10-year high water elevation including wave height.</td></tr>
</table>

GEOMETRIC DESIGN CRITERIA FOR LOCAL URBAN STREETS
(New Construction/Reconstruction)
Figure 14.3-C

(Continued on next page)

Footnotes for Figure 14.3-C

(1) Design Forecast Year. Table values are desirable. For urban streets, the minimum design year is 10 years.

(2) Minimum Design Speed. Design speed is not a major factor for Group 1 roads and streets. Select a design speed based on available right of way, terrain, likely pedestrian presence, adjacent development and other area controls.

(3) Shoulder Width. Shoulders should be increased by 3.5 feet where guardrail is used.

(4) Auxiliary Lane Width. The auxiliary lane width should be the same as the adjacent travel lane.

(5) Shoulder Cross Slope. For paved shoulders wider than 4 feet, use a 4.00 percent shoulder cross slope.

(6) Bicycle Facilities Lane Width. If curb and gutter is provided, provide a 4-foot width from the face of curb. For design speeds greater than 45 miles per hour, increase the bike lane width in accordance with AASHTO *Guide for the Development of Bicycle Facilities*.

(7) Curb and Gutter (Type). If curb and gutter is used on streets with design speeds greater than 45 miles per hour, place the curb and gutter outside of the shoulder and use a sloping curb. In some residential areas, an OGEE curb and gutter may be used.

(8) Clear Zones. See the AASHTO *Roadside Design Guide for the applicable clear zones.*

(9) Bridge Widths. Bridge width is equal to width of roadway section (outside shoulder to outside shoulder). See Section 7.5.1.1 for guidance.

(10) Structural Capacity (Existing Bridges). Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(11) Vertical Clearance (Local Roads Under).

a. Provide the vertical clearance over the entire traveled way, shoulders and any anticipated future widening.

b. Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR LOCAL URBAN STREETS
(New Construction/Reconstruction)
Figure 14.3-C
(Continued)

Design Element		Manual Section	Design Speed								
			20 mph	25 mph	30 mph	35 mph	40 mph	45 mph	50 mph	55 mph	60 mph
Stopping Sight Distance **(1)**		4.1	115 ft	155 ft	200 ft	250 ft	305 ft	360 ft	425 ft	495 ft	570 ft
Decision Sight Distance **(2)**		4.3	-	-	490 ft	590 ft	330 ft	690 ft	800 ft	910 ft	1030
Intersection Sight Distance **(3)**		4.4	225 ft	280 ft	335 ft	390 ft	445 ft	500 ft	555 ft	610 ft	665 ft
Minimum Radii	e_{max} = 4%		86 ft	154 ft	250 ft	371 ft	533 ft	711 ft	926 ft	1190	1500
Superelevation Rate **(4)**		5.3	4%	4%	4%	4%	4%	4%	4%	4%	4%
Horizontal Sight Line Offset **(5)**		5.4	19 ft	19 ft	20 ft	21 ft	22 ft	23 ft	24 ft	26 ft	27 ft
Min. Vertical (K-values) **(6)**	Crest	6.5	7	12	19	29	44	61	84	114	151
	Sag		17	26	37	49	64	79	96	115	136
Maximum Grade **(7)**	Level	6.3.1	8%	7%	7%	7%	7%	6%	6%	6%	5%
	Rolling		11%	11%	10%	10%	9%	8%	7%	7%	6%
	Mountain		15%	15%	14%	13%	12%	10%	10%	10%	n/a
Minimum Grade **(8)**		6.3.2	Des.: 0.5% Min.: 0.3%								

<u>Footnotes</u>

(1) <u>Stopping Sight Distance</u>. Table values are for passenger cars on level grade.

(2) <u>Decision Sight Distance</u>. Table values are for stop on an urban road, Avoidance Maneuver B. See Section 4.3 for other maneuvers.

(3) <u>Intersection Sight Distance</u>. Table values are for passenger cars for assumed conditions described in Figure 4.4-C. See Section 4.4 for other conditions.

(4) <u>Superelevation Rate</u>. See Section 5.3 for superelevation rates based on e_{max}, design speed and radii of horizontal curves.

(5) <u>Horizontal Sight Line Offset</u>. Table values provide the necessary middle ordinate assuming the design speed, stopping sight distance and minimum radii based on an e_{max} = 4 percent.

(6) <u>Vertical Curvature (K-Value)</u>. K-values are based on the level stopping sight distances.

(7) <u>Maximum Grades</u>. For urban streets in commercial and industrial areas, limits grades to 8 percent or less.

(8) <u>Minimum Grade</u>. The minimum for curb and gutter is 0.3 percent and for valley gutter it is 0.4 percent. Special ditch grades may be necessary to ensure proper project runoff management.

ALIGNMENT CRITERIA FOR LOCAL URBAN STREETS
(New Construction/Reconstruction)
Figure 14.3-D

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14.4 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. *Highway Safety Design and Operations Guide*, AASHTO, 1997.

3. *Roadside Design Guide*, AASHTO, 2012.

4. *Highway Capacity Manual (HCM) 2010*, Transportation Research Board, 2010.

5. *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT $\leq$ 400)*, AASHTO, 2001.

6. *Guide for the Development of Bicycle Facilities*, AASHTO, 2012.

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Chapter 15

COLLECTOR ROADS AND STREETS

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Section **Page**

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Chapter 15
COLLECTOR ROADS AND STREETS

This chapter discusses the minimum criteria used in the design of collector roads and streets. Information that is also applicable to the design of collector roads and streets is included in the following chapters:

- Chapter 3 “Basic Design Controls,” Chapter 4 “Sight Distance,” Chapter 5 “Horizontal Alignment,” Chapter 6 “Vertical Alignment” and Chapter 7 “Cross Section Elements” provide guidance on geometric design elements.

- Chapter 9 “Intersections” provides information on the design of intersections, including intersection alignment, left- and right-turn lanes and channelization.

15.1 FUNCTIONAL CLASSIFICATION

Collector routes are characterized by a roughly even distribution of their access and mobility functions. Traffic volumes and speeds will typically be somewhat lower than those of arterials. Access to properties is normally allowed on collector roads. Section 3.4.1 provides guidance on functional classifications.

The function of rural collector roads is to serve intracounty travel needs and collect traffic flow from the rural local roads to the rural arterials and to distribute traffic flow from arterials back to the local roads. In rural areas, the collectors provide the following functions:

- provide access to adjacent land uses;

- carry traffic into areas with sparse development;

- serve larger towns and significant traffic generators (e.g., shipping ports, mining areas) that are not served by an arterial or freeway;

- spaced at intervals consistent with the traffic population density to accumulate traffic from local roads;

- provide service to smaller communities; and

- link locally important traffic generators with higher classified routes.

In urban areas, collector streets serve as intermediate links between the arterial system and points of origin and destination. Urban collectors typically have the following characteristics:

- provide both access and traffic circulation within residential neighborhoods and commercial/industrial areas;

- may penetrate residential neighborhoods or commercial/industrial areas to collect and distribute trips to and from the arterial system;

- in the Central Business District (CBD), may include the streets that are not classified as arterials;
- in fully developed areas, spacing generally is approximately ½ mile between routes and, within the CBD, between 650 feet and ½ mile;
- may be an urban extension of rural collector roads; and
- often include local bus routes.

To determine the functional classification of a facility, the designer should contact Road Data Services.

15.2 DESIGN ELEMENTS

15.2.1 Traffic Volumes

Traffic volumes are a major consideration in justifying highway facilities and assisting designers in the establishment of geometric and cross section design characteristics. The designer should use the design year traffic volumes to determine the design criteria for collector roads and streets.

For urban streets, traffic volumes and characteristics usually dominate vehicular traffic demands. In addition, the designer must also consider pedestrians, bicyclists and transit service. For urban streets, the designer should determine the annual average daily traffic (AADT), peak-hour traffic, peak-hour factor, directional distribution, traffic composition and projection of future traffic demands for all modes of travel. The designer should review the *Highway Capacity Manual* for guidance on making these determinations.

15.2.2 Level of Service

Design the highway mainline and intersections to accommodate the selected design hourly volume (DHV) at the selected level of service (LOS). This may involve adjusting the various highway factors that affect capacity until the design will accommodate the DHV. Further discussion on the LOS design concept is included in Section 3.6.4. Detailed calculations, factors and methodologies are presented in the *Highway Capacity Manual*.

15.2.3 Design Speed

The design speed establishes the range of design values for many of the geometric elements of the highway (e.g., sight distance, horizontal alignment, vertical alignment). The selected design speed should be high enough so that an appropriate regulatory speed limit will be less than or equal to it. Desirably, the speed at which drivers are operating comfortably will be close to the posted speed limit. See Section 3.5.2 and the FHWA publication *Mitigation Strategies for Design Exceptions* for additional guidance on the selection of design speeds.

Design speeds for rural collectors are based on terrain, traffic volumes, driver expectancy and alignment, and may range from 30 to 60 miles per hour. Urban design speeds for collectors can range from 30 to 45 miles per hour, depending on available right of way, terrain, adjacent development, likely pedestrian presence and other site controls. Design speeds in CBDs are generally 30 miles per hour or less, while higher speeds are more applicable to outlying suburban and developing areas.

The geometric design tables in Section 15.3 provide the applicable design speeds for collector roads and streets.

15.2.4 Sight Distances

See Chapter 4 “Sight Distance” for guidance on stopping, decision, passing and intersection sight distances.

15.2.5 Alignment

The horizontal and vertical alignment should complement each other and should be considered in combination to achieve appropriate safety, capacity and appearance for the type of improvement proposed. Proper combinations of curvature, tangents, grades, variable median widths and separate roadway elevations all combine to enhance safety and aesthetics of collectors. When designing the horizontal and vertical alignments, the designer should consider the following:

1. Horizontal Alignment. Note the following:

    a. Rural Collectors. The designer should provide the most favorable alignment practical for rural collectors. The following guidelines should be applied when laying out the horizontal alignment:

        - Only use minimum radii where it is necessary due to restricted conditions.
        - Avoid abrupt changes in alignment.
        - Avoid alignments that require superelevation transitions on bridges, bridge approach slabs or at intersections.

    b. Low-Speed Urban Collectors. Where superelevation is required on low-speed urban streets (V_d ≤ 45 mph), the design should use AASHTO Method 2 in determining the design superelevation. See Chapter 5 “Horizontal Alignment” for minimum radii and superelevation rates for low-speed urban streets.

2. Vertical Alignment. Even though the profile may satisfy all design controls, the use of minimum criteria may appear forced and angular. Therefore, the designer should use higher values to produce a smoother, more aesthetically pleasing alignment. Note that flat vertical curves may produce flat areas that may cause drainage problems. For further guidance, see Chapter 6 “Vertical Alignment.”

3. Horizontal and Vertical Combinations. Consider the relationship between horizontal and vertical alignments simultaneously to obtain a desirable condition. Section 6.2.2 discusses this relationship in detail and its effect on aesthetics and safety.

4. Minimum Grades. Desirably, the longitudinal grade should be 0.5 percent or greater. For curbed facilities and bridges, it is necessary to provide a minimum longitudinal grade of 0.3 percent to facilitate drainage. For curbed sections, ensure curb profiles provide positive drainage. For uncurbed facilities, a minimum longitudinal grade of 0.0 percent may be considered if adequate cross slopes are provided. Ensure superelevation transitions are not developed in areas with 0.0 percent grade. Special ditch grades may be necessary to ensure proper drainage.

5. Climbing Lanes. Section 6.4 discusses the warrants and design criteria for climbing lanes.

15.2.6 Cross Section Elements

The following sections summarize the cross section criteria for collectors. For additional information concerning cross sections, the designer should review Chapter 7 “Cross Section Elements.”

15.2.6.1 Typical Sections

The following figures present typical sections for collector roads and streets:

- Figure 15.2-A – Typical Rural Two-Lane Collector
- Figure 15.2-B – Typical Urban Five-Lane Collector (TWLTL) with Shoulders
- Figure 15.2-C – Typical Urban Two-Lane Collector (Curb and Gutter and Bike Lanes)

15.2.6.2 Travel Lane and Shoulder Widths

Travel lane widths on rural collectors should be 11 to 12 feet. For rural collectors where the ADT is less than 250 vehicles per day and where the design speed is 40 miles per hour or less, the designer may consider a 10-foot travel lane. Travel lane widths for urban collectors should be 12 feet; however, travel lane widths in CBDs may be 11 feet if the truck traffic is less than or equal to 5 percent.

Provide a 6-foot shoulder where the ADT is 2000 vehicles per day or less and an 8-foot shoulder for facilities with greater ADTs. The shoulder width includes a minimum paved width of 2 feet. Where bicycles are to be accommodated on the shoulder, the designer should provide a minimum paved shoulder width of 4 feet. In constrained urban areas with curb and gutter and low speeds, the shoulder width may be just the 2-foot curb and gutter width. On high-speed facilities with curb and gutter sections, provide an 8-foot shoulder.

15.2.6.3 Cross Slopes

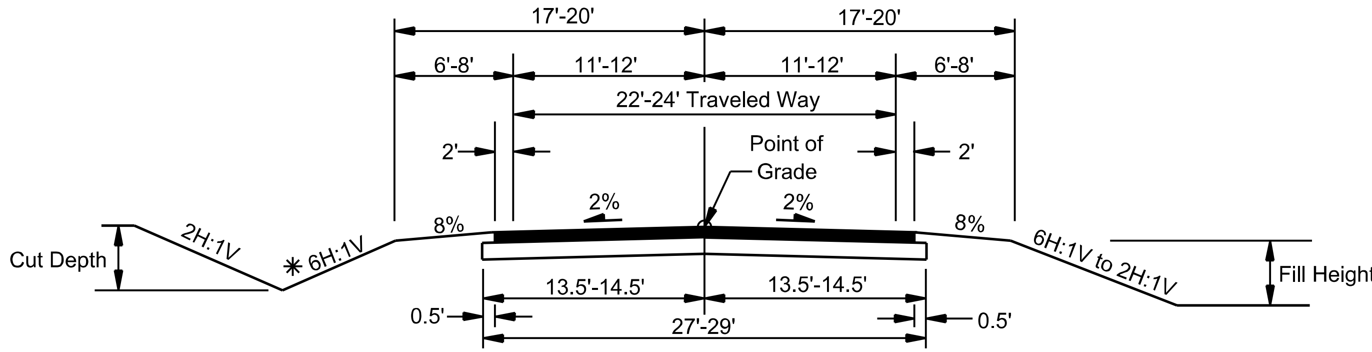
Use a cross slope of 2.00 percent for up to two lanes plus one half the width of the flush median or TWLTL. Travel lanes beyond the second lane on one side of the crown should have a cross slope of 2.50 percent. Crown the pavement at the center of the TWLTL and use a cross slope of 2.00 percent away from the centerline for all lanes on three- and five-lane highways. For a seven-lane section, use a cross slope of 2.50 percent for the outside lanes. If a roadway profile grade is less than 2.00 percent, the designer may consider using a cross slope of 2.50 percent for the outside lane to improve drainage. See Section 7.2.3.3.

For paved shoulders greater than 4 feet, provide a shoulder cross slope of 4.00 percent. For paved shoulders less than or equal to 4 feet, the cross slope should match the adjacent travel lane slope. For earth shoulders, provide a shoulder cross slope of 8.00 percent.

For cross slopes on bridges, see the *SCDOT Bridge Design Manual*.

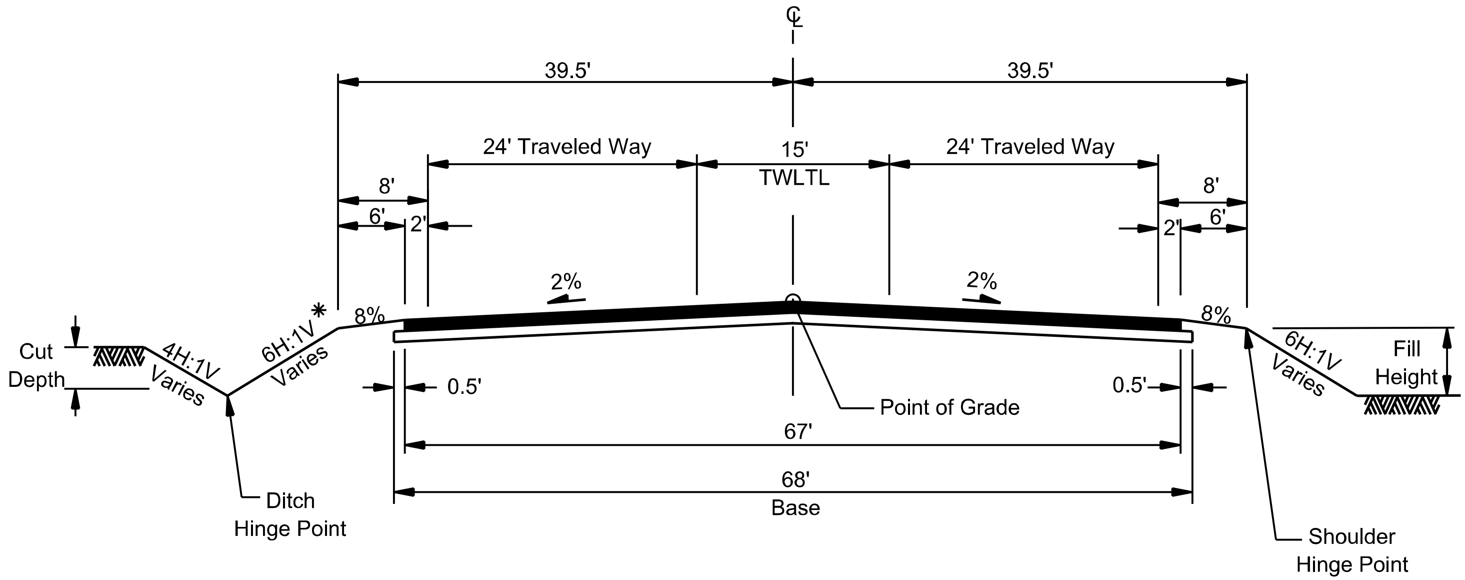
15.2.6.4 Auxiliary Lanes

Auxiliary lanes (e.g., passing lanes, parking lanes, turn lanes) are lanes beyond the through travel lanes intended for use by vehicular traffic for specific functions. Desirably, auxiliary lanes will have the same width and cross slope as the adjacent through lanes, although in many cases a lesser width may be appropriate. The geometric design tables in Section 15.3 present lane and shoulder widths for auxiliary lanes.



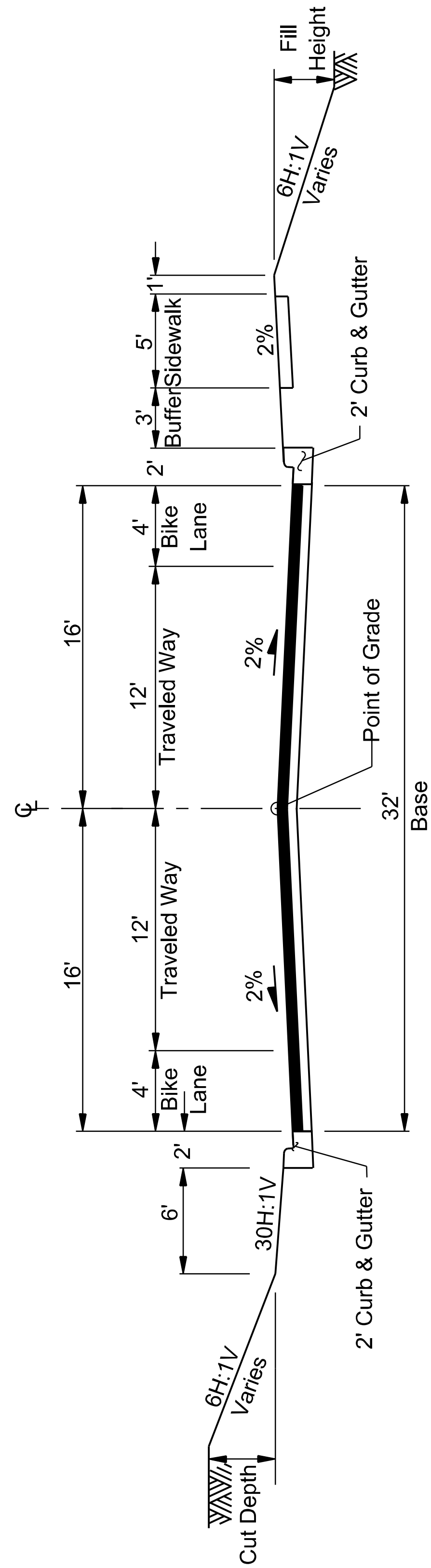
*This slope may vary between a minimum slope of 12H:1V to a maximum slope of 4H:1V. Where a deeper ditch than provided by a 4H:1V slope is necessary for drainage purposes, continue the 4H:1V slope until the necessary depth has been obtained. This will place the ditch further from the roadway. Provide a separate profile for special ditch grades.

TYPICAL RURAL TWO-LANE COLLECTOR
Figure 15.2-A



*This slope may vary between a minimum slope of 12H:1V to a maximum slope of 4H:1V. Where a deeper ditch than provided by a 4H:1V slope is necessary for drainage purposes, continue the 4H:1V slope until the necessary depth has been obtained. This will place the ditch further from the roadway. Provide a separate profile for special ditch grades.

TYPICAL URBAN FIVE-LANE COLLECTOR (TWLTL) WITH SHOULDERS
Figure 15.2-B



TYPICAL URBAN TWO-LANE COLLECTOR
(Curb and Gutter and Bike Lanes)
Figure 15.2-C

15.2.6.5 Bicycle Accommodations

For accommodation of bicyclists, the designer should review the guidance provided in Section 11.11. In general, the designer should provide a minimum 4-foot paved shoulder for shared roadways or a minimum 4-foot bike lane.

15.2.6.6 Medians

A median may be considered on an urban collector. The principal functions of a median are to provide separation from opposing traffic, manage access, accommodate turning movements, provide a pedestrian refuge and to allow additional width for future lanes. Medians on urban collectors may be one of the following median types:

1. <u>Flush Medians</u>. Flush medians provide an area for left-turn movements and permit direct access to adjoining properties. This allows for numerous unrestricted conflict points. The flush median may serve as refuge for disabled vehicles and as a temporary lane for emergency vehicles. The two-way, left-turn lane (TWLTL) is considered a type of flush median. Desirably, the roadway cross section with a flush median will allow development of a TWLTL, if applicable.

2. <u>Raised Medians</u>. Raised medians restrict left-turn movements to select locations, which allows for better access management. This median may provide a refuge area for pedestrians and an open space for aesthetic considerations.

For guidance on medians and TWLTL, see Chapter 7 “Cross Section Elements.”

15.2.6.7 Right of Way

Providing right-of-way widths that accommodate construction, drainage and proper maintenance of a collector is an important part of the overall design. Wider right of way allows for gentler side slopes, which results in reduced crash severity potential and easier maintenance operations. Right of way is typically configured to accommodate all proposed cross section elements throughout the project (e.g., travel lanes, shoulders, medians, parking lanes, bike lanes, sidewalks, ditches, outer slopes). If a long-range plan identifies a future widening, give consideration to accommodate a future proposed cross section. A uniform right-of-way width is preferred; however, do not base the width on the critical point of the project. A critical point may occur where the side slopes extend beyond the normal right of way, for clear areas at the bottom of traversable slopes, for wider clear areas on the outside of curves, where greater sight distance is desirable, at intersections and junctions with other roads, at railroad-roadway grade crossings, for environmental considerations and for maintenance access.

15.2.7 <u>Alternatives to Widening Two-Lane Facilities</u>

For rural two-lane collectors that are not candidates for widening to a four-lane or five-lane facility, but are experiencing operational and safety problems or site-specific reductions in the level of service, the designer should consider the guidance provided in Section 16.2.7 for possible improvements.

15.2.8 Roadside Safety

The designer should provide adequate horizontal clearance between the traveled way and roadside obstructions on collectors. The designer should provide roadside clear zones as discussed in the AASHTO *Roadside Design Guide*.

15.3 TABLES OF DESIGN CRITERIA

The geometric design tables in this section present the Department's design and alignment criteria for rural and urban collector projects. The designer should consider the following when using these tables:

1. Applicability. Note that some cross-section elements included in the tables (e.g., bike lanes) are not automatically warranted in the project design. The values in the figures only apply after the decision has been made to include the design element in the highway cross section.

2. Manual Section References. These tables are intended to provide a concise listing of design values for easy use. However, the designer should review the *Highway Design Manual* section references for more information on the design elements.

3. Footnotes. The figures include many footnotes, which are identified by a number in parentheses (e.g., **(3)**). The information in the footnotes is critical to the proper use of the design tables.

The following design tables are provided for collectors:

- Figure 15.3-A — "Geometric Design Criteria for Rural Collectors (New Construction/ Reconstruction)"
- Figure 15.3-B — "Alignment Criteria for Rural Collectors (New Construction/ Reconstruction)"
- Figure 15.3-C — "Geometric Design Criteria for Urban Collectors (New Construction/ Reconstruction)"
- Figure 15.3-D — "Alignment Criteria for Urban Collectors (New Construction/ Reconstruction)"

Design Element				**Manual Section**	**Rural**			
Design Controls	Design Year Traffic (AADT)			3.6.3	**≤ 400**	**401 to 1500**	**1501 to 2000**	**Over 2000**
	Design Forecast Year			15.2.1	20 Years			
	Minimum Design Speed	Level		15.2.3	40 mph	50 mph	50 mph	60 mph
		Rolling			30 mph	40 mph	40 mph	50 mph
		Mountainous			30 mph	30 mph	30 mph	40 mph
	Access Control			3.8	Control by Regulation			
	Level of Service			3.6.4	Level/Rolling: C Mountainous: D			
Cross Section Elements	Travel Lane Width			15.2.6	11 ft **(1a)**	11 ft	11 ft	12 ft **(1b)**
	Shoulder **(2)**	Total Width		15.2.6	6 ft	6 ft	6 ft	8 ft
		Paved Width			2 ft			
	Auxiliary Lanes	Lane Width		15.2.6	Same as Mainline Travel Lanes			
		Shoulder Width	Total		Same as Mainline Shoulders			
			Paved		2 ft			
	Cross Slope	Travel Lane		15.2.6	2.00%			
		Auxiliary Lane			2.00%			
		Shoulder	Paved **(3)**		2.00%			
			Unpaved		8.00%			
	TWLTL			15.2.6	15 ft			
	Rights of Way Width			15.2.6	Project Specific			
Roadway Slopes	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V to 4H:1V			
			Ditch Type		V-Ditch			
			Back Slope		4H:1V to 2H:1V			
			Rock Cut		0.25H:1V			
		Fill Section	0 ft – 5 ft	7.3.2	6H:1V			
			5 ft – 10 ft		4H:1V			
			> 10 ft		2H:1V			
	Clear Zone				**(4)**			

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS
(New Construction/Reconstruction)
Figure 15.3-A
(Continued on next page)

Design Element			Manual Section	Rural			
Structures	New and Reconstructed Bridges	Structural Capacity	7.5.1	HL-93			
		Clear Roadway Width **(5)**		34 ft	34 ft	34 ft	40 ft
	Existing Bridges to Remain in Place	Structural Capacity	7.5.1	**(6)**			
		Clear Roadway Width (**5)**		22 ft	22 ft	24 ft	28 ft
	Vertical Clearance (Collector Under) **(7a)**	New and Replaced Overpassing Bridges **(7b)**	6.6	16 ft – 0 in			
		Existing Overpassing Bridges		16 ft – 0 in			
		Pedestrian Bridges		18 ft – 0 in			
		Overhead Signs		17 ft – 6 in			
		Overhead Utilities		Coordinate with Utilities Office			
	Vertical Clearance (Collector Over)	Railroads	6.6	23 ft – 0 in			
		Underpass Width	7.5.2	Traveled Way plus Clear Zone			
	Vertical Clearance (Over Water)	Navigable Water	6.6	See Environmental Services Office			
		Major Lakes & Reservoirs (with boat traffic)		8 ft – 0 in above the high water mark			
		Rivers		2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.			
		Tidal Waters		2 ft above the 10-year high water elevation including wave height.			

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS
(New Construction/Reconstruction)
Figure 15.3-A
(Continued on next page)

<u>Footnotes for Figure 15.3-A</u>

(1) <u>Travel Lane Width</u>.

a. Where the design speed is 40 miles per hour or less and the ADT is less than 250 vehicles per day, 10-foot travel lanes may be considered.

b. On reconstructed collectors, an existing 22-foot traveled way may be retained where the alignment is satisfactory and there is no crash pattern suggesting the need for widening.

(2) <u>Shoulder Width (Total Width)</u>. Where guardrail is required, increase the shoulder width an additional 3.5 feet.

(3) <u>Shoulder Cross Slope</u>. For paved shoulders wider than 4 feet, use a 4.00 percent shoulder cross slope.

(4) <u>Clear Zone</u>. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(5) <u>Bridge Widths</u>. Clear roadway bridge widths are measured from face to face of parapets or rails. Bridge widths are normally defined as the sum of the approach traveled way width plus total width for both shoulders. See Section 7.5.1.1 for further guidance.

(6) <u>Structural Capacity (Existing Bridges)</u>. Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(7) <u>Vertical Clearance (Collector Under)</u>.

a. The clearance must be available over the traveled way, shoulders and any future widening identified in a long-range plan.

b. Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTORS
(New Construction/Reconstruction)
Figure 15.3-A
(Continued)

Design Element		Manual Section	Design Speed						
			30 mph	**35mph**	**40 mph**	**45 mph**	**50 mph**	**55 mph**	**60 mph**
Stopping Sight Distance **(1)**		4.1	200 ft	250 ft	305 ft	360 ft	425 ft	495 ft	570 ft
Passing Sight Distance		4.2	500 ft	550 ft	600 ft	700 ft	800 ft	900 ft	1000 ft
Decision Sight Distance **(2)**		4.3	450 ft	525 ft	600 ft	675 ft	750 ft	865 ft	990 ft
Intersection Sight Distance **(3)**		4.4	335 ft	390 ft	445 ft	500 ft	555 ft	610 ft	665 ft
Minimum Radii	e_{max} = 8%	5.2					758 ft	960 ft	1200 ft
	e_{max} = 6%		231 ft	340 ft	485 ft	643 ft	833 ft		
Superelevation Rate **(4)**		5.3	6%	6%	6%	6%	6% or 8%	8%	8%
Horizontal Sight Line Offset **(5)**		5.4	21 ft	23 ft	24 ft	25 ft	27/30 ft	32 ft	34 ft
Vertical Curvature (K-Values) **(6)**	Crest	6.5	19	29	44	61	84	114	151
	Sag		37	49	64	79	96	115	136
Maximum Grade **(7)**	Level	6.3.1	7%	7%	7%	7%	6%	6%	5%
	Rolling		9%	9%	8%	8%	7%	7%	6%
	Mountainous		10%	10%	10%	10%	9%	9%	8%
Minimum Grade **(8)**		6.3.2	0.5%						

Footnotes

(1) Stopping Sight Distance. Table values are for passenger cars on level grade.

(2) Decision Sight Distance. Table values are for speed/path/direction change on rural road, Avoidance Maneuver C. See Section 4.3 for other maneuvers.

(3) Intersection Sight Distance. Table values are for passenger cars for assumed conditions described in Figure 4.4-C. See Section 4.4 for other conditions.

(4) Superelevation Rate. See Section 5.3 for superelevation rates based on e_{max}, design speed and radii of horizontal curves.

(5) Horizontal Sight Line Offset. Table values provide the necessary middle ordinate assuming the design speed, stopping sight distance and minimum radii based on an e_{max} = 6 percent for design speeds 30 to 50 miles per hour and e_{max} = 8 percent for design speeds of 50 to 60 miles per hour.

(6) Vertical Curvature (K-Value). K-values are based on the level stopping sight distances.

(7) Maximum Grade. Short lengths of grades (e.g., less than 500 feet), one-way downgrades and low-volume collectors may be up to 2 percent steeper.

(8) Minimum Grade. Longitudinal gradients of 0.0 percent may be acceptable on some pavements that have cross slopes that have adequate drainage. Ensure superelevation transitions are not developed in areas with 0.0 percent grade. Special ditch grades may be necessary to ensure proper project runoff management.

ALIGNMENT CRITERIA FOR RURAL COLLECTORS
(New Construction/Reconstruction)
Figure 15.3-B

Design Element				Manual Section	Urban
Design Controls	Design Forecast Year			15.2.1	20 Years
	Minimum Design Speed			15.2.3	30 mph
	Access Control			3.8	Limited/Control by Regulation
	Level of Service			3.6.4	Desirable: C
Cross Section Elements	Travel Lane Width **(1)**			15.2.6	12 ft
	Shoulder Width	Total		15.2.6	8 ft or Curb and Gutter
		Paved			2 ft or Curb and Gutter
	Auxiliary Lanes	Lane Width		15.2.6	match travel lane
		Shoulder Width	Total		match travel lane shoulder
			Paved		2 ft or Curb and Gutter
	Parking Lane Width			7.2.7	8-12 ft
	Cross Slope	Travel Lane		15.2.6	2.00%
		Auxiliary Lane			2.00%
		Shoulder	Paved **(2)**		2.00%
			Unpaved		8.00%
	Bicycle	Bike Lane Width **(3)**		11.11	4 ft
		Shared Roadway Width			14 ft Outside Travel Lane
	Curb and Gutter	Type **(4)**		7.2.8	Vertical or Sloping
		Width			2 ft
	Sidewalk Width			7.3.3	5 ft
	Median	Width	Flush	15.2.6	Desirable: 12 ft Minimum: 4 ft
			TWLTL		15 ft
		Flush/TWLTL Slopes		7.3.2	2.00%
	Right of Way Width			15.2.6	Project Specific
Roadway Slopes	Side Slopes	Cut Section	Foreslope	7.3.2	12.5H:1V to 4H:1V
			Ditch Type		V-Ditch
			Back Slope		4H:1V to 2H:1V
		Fill Section	0 ft – 5 ft	7.3.2	6H:1V
			5 ft – 10 ft		4H:1V
			> 10 ft		2H:1V
	Clear Zone				**(5)**

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTORS
(New Construction/Reconstruction)
Figure 15.3-C

(Continued on next page)

Design Element			Manual Section	Urban
Structures	New Bridges	Structural Capacity	7.5.1	HL-93
		Clear Roadway Width		**(6)**
	Existing Bridges to Remain in Place	Structural Capacity	7.5.1	**(7)**
		Clear Roadway Width		**(6)**
	Vertical Clearance (Collector Under) **(8a)**	New and Replaced Overpass Bridges **(8b)**	6.6	16 ft – 0 in
		Existing Overpassing Bridges	6.6	16 ft – 0 in
		Pedestrian Bridges	6.6	18 ft – 0 in
		Overhead Signs	6.6	17 ft – 6 in
		Overhead Utilities	6.6	Coordinate with Utility Office
	Vertical Clearance (Collector Over)	Railroads	6.6	23 ft – 0 in
		Underpass Width	7.5.2	Traveled Way plus Clear Zone
	Vertical Clearance (Over Water)	Navigable Water	6.6	See Environmental Services Office
		Major Lakes & Reservoirs (with boat traffic)		8 ft – 0 in above the high water mark
		Rivers		2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.
		Tidal Waters		2 ft above the 10-year high water elevation including wave height.

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTORS
(New Construction/Reconstruction)
Figure 15.3-C
(Continued on next page)

Footnotes for Figure 15.3-C

(1) Travel Lane Width. In CBDs, an 11-foot traveling lane may be used if the truck volumes are less than or equal to 5 percent.

(2) Shoulder Cross Slope. For paved shoulders wider than 4 feet, use a 4.00 percent shoulder cross slope.

(3) Bicycle (Lane Width). For design speeds greater than 45 miles per hour, increase the bike lane width in accordance with Section 11.11 and the AASHTO *Guide for the Development of Bicycle Facilities*.

(4) Curb and Gutter (Type). If curb and gutter is used on streets with design speeds greater than 45 miles per hour, place the curb and gutter outside of the shoulder and use a sloping curb.

(5) Clear Zone. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(6) Bridge Widths. Clear roadway bridge widths are measured from face to face of parapets or rails. Bridge widths are normally defined as the sum of the approach traveled way width, shoulders and median width, where applicable. For curbed sections, the clear roadway width will be the curb-to-curb width plus the sidewalk width on one or both sides. See Section 7.5.1.1 for further guidance.

(7) Structural Capacity (Existing Bridges). Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(8) Vertical Clearance (Collector Under).

a. The clearance must be available over the traveled way, shoulders and any future widening identified in a long-range plan.

b. Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTORS
(New Construction/Reconstruction)
Figure 15.3-C
(Continued)

Design Element		Manual Section	Design Speed			
			30 mph	**35 mph**	**40 mph**	**45 mph**
Stopping Sight Distance **(1)**		4.1	200 ft	250 ft	305 ft	360 ft
Decision Sight Distance **(2)**		4.3	490 ft	590 ft	690 ft	800 ft
Intersection Sight Distance **(3)**		4.4	335 ft	390 ft	445 ft	500 ft
Minimum Radii	e_{max} = 6%	5.2	231 ft	340 ft	485 ft	643 ft
	e_{max} = 4%		250 ft	371 ft	533 ft	711 ft
Superelevation Rate **(4)**		5.3	4% or 6%	4% or 6%	4% or 6%	4% or 6%
Horizontal Sight Line Offset **(5)**		5.4	21 ft	23 ft	24 ft	25 ft
Vertical Curvature (K-Values) **(6)**	Crest	6.5	19	29	44	61
	Sag		37	49	64	79
Maximum Grade **(7)**	Level	6.3.1	9%	9%	9%	8%
	Rolling		11%	10%	10%	9%
	Mountainous		12%	12%	12%	11%
Minimum Grade		6.3.2	Desirable: 0.5% Minimum: 0.3% (Curb and Gutter)			

Footnotes

(1) Stopping Sight Distance. Table values are for passenger cars on level grade.

(2) Decision Sight Distance. Table values are for stop on an urban road, Avoidance Maneuver B, as described in Figure 4.3-A.

(3) Intersection Sight Distance. Table values are for passenger cars for assumed conditions described in Figure 4.4-C. See Section 4.4 for other conditions.

(4) Superelevation Rate.

a. See Section 5.3 for superelevation rates based on e_{max}, design speed and radii of horizontal curves.

b. The 6 percent superelevation rate should only be used on suburban collectors.

(5) Horizontal Sight Line Offset. Table values provide the necessary middle ordinate assuming the design speed, stopping sight distance and minimum radii based on an e_{max} = 6 percent.

(6) Vertical Curvature (K-Value). K-values are based on the level stopping sight distances.

(7) Maximum Grade. Short lengths of grades (e.g., less than 500 feet), one-way downgrades and low-volume collectors may be up to 2 percent steeper.

ALIGNMENT CRITERIA FOR URBAN COLLECTORS
(New Construction/Reconstruction)
Figure 15.3-D

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15.4 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.
2. *Mitigation Strategies for Design Exceptions*, FHWA, 2007.
3. *Highway Safety Design and Operations Guide*, AASHTO, 1997.
4. *Roadside Design Guide*, AASHTO, 2012.
5. *Highway Capacity Manual*, Transportation Research Board, 2010.

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Chapter 16

RURAL AND URBAN ARTERIALS

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Table of Contents

Section **Page**

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Chapter 16
RURAL AND URBAN ARTERIALS

This chapter discusses the minimum criteria used in the design of arterial roads and streets. Information that is also applicable to the design of rural and urban arterials is included in the following chapters:

- Chapter 3 “Basic Design Controls,” Chapter 4 “Sight Distance,” Chapter 5 “Horizontal Alignment,” Chapter 6 “Vertical Alignment” and Chapter 7 “Cross Section Elements” provide guidance on geometric design elements.
- Chapter 9 “Intersections” provides information on the design of intersections, including intersection alignment, left- and right-turn lanes and channelization.
- Chapter 10 “Interchanges” discusses the type, location and design of interchanges.

16.1 FUNCTIONAL CLASSIFICATION

Chapter 16 addresses arterial highways, see Section 3.4.1 for additional guidance on functional classifications.

Arterial highways are generally characterized by their ability to quickly move relatively large volumes of traffic, but is often impacted by access to abutting properties. The arterial system typically provides for high travel speeds and the longest trip movements. The rural and urban arterial systems are connected to provide continuous through movements at approximately the same level of service.

The freeway is the highest level of an arterial. These facilities are characterized by full control of access, high design speeds and a high level of driver comfort and safety. For these reasons, freeways are considered a special type of highway within the functional classification system, and separate design criteria have been developed for freeways in Chapter 17 “Freeways.”

Arterials have the following general characteristics:

- consist of a connected network of continuous routes;
- in rural areas, provide a mix of interstate and intercounty travel service;
- provide service to, through or around urban areas from rural arterial routes and may be connecting links;
- provide for significant urban and suburban travel demands (e.g., between central business districts (CBD) and outlying residential areas, between major inner city communities, between major suburban centers);

- serve long distance traffic within an urban area by connecting major regional activity centers not served by connecting links or may provide service for trips of moderate length;
- may be a multilane undivided facility, divided facility, two-lane rural highway, major two-way city street or a one-way pair system;
- typically warrant management of access to the highway;
- may be included in the National Highway System (NHS); and
- may carry local bus routes and provide intra-community continuity, but generally will not penetrate neighborhoods.

To determine the functional classification of a facility, the designer should contact Road Data Services.

16.2 DESIGN ELEMENTS

16.2.1 Traffic Volumes

Traffic volumes are a major consideration in justifying highway facilities and assisting designers in the establishment of geometric and cross section design characteristics. The designer should use design-year traffic volumes to determine design elements of rural and urban arterials.

For urban streets, traffic volumes and characteristics usually dominate vehicular traffic demands. In addition, the designer must also consider pedestrians, bicyclists and transit service. For urban streets, the designer should determine the Annual Average Daily Traffic (AADT), peak-hour traffic, peak-hour factor, directional distribution, traffic composition and projection of future traffic demands for all modes of travel. The designer should review the *Highway Capacity Manual* for guidance on making these determinations.

16.2.2 Level of Service

Design the highway mainline, intersection or interchange to accommodate the selected design hourly volume (DHV) at the selected level of service (LOS). This may involve adjusting the various highway factors that affect capacity until a design is determined that will accommodate the DHV. Further discussion on the LOS design concept is included in Section 3.6.4. Detailed calculations, factors and methodologies are presented in the *Highway Capacity Manual*.

16.2.3 Design Speed

Design speed is a selected speed used to determine the various design features of the roadway. Design speeds for rural arterials are based on terrain, driver expectancy and the alignment. See Section 3.5.2 and the FHWA publication *Mitigation Strategies for Design Exceptions* for additional guidance on the selection of design speeds.

Urban arterial design speeds can vary from 30 mph to 60 mph, depending on available right of way, terrain, adjacent development, likely pedestrian presence and other site controls. Lower speeds apply in CBD and in more developed areas, while higher speeds are more applicable to outlying suburban and developing areas.

The geometric design tables in Section 16.3 provide the applicable design speeds for rural and urban arterials.

16.2.4 Sight Distances

See Chapter 4 “Sight Distance” for guidance on stopping, decision, passing and intersection sight distances.

16.2.5 Alignment

Designed for high-volume and high-speed operations, arterials should have smooth horizontal and vertical alignments. Proper combinations of curvature, tangents, grades, variable median widths and separate roadway elevations all combine to enhance safety and aesthetics of arterials. When designing arterial alignments, the designer should consider the following:

1. Horizontal Alignment. Note the following:

    a. Rural Arterials. The following guidelines should be applied when laying out the horizontal alignment:

        - Avoid the use of minimum length of curve.
        - Only use minimum radii where it is necessary due to restricted conditions.
        - Avoid alignments that require superelevation transitions on bridges, bridge approach slabs or intersections.

    b. Low-Speed Urban Arterials. Where superelevation is required on low-speed urban streets ($V_d \leq 45$ mph), use AASHTO Method 2 in determining the design superelevation. See Chapter 5 "Horizontal Alignment" for minimum radii and superelevation rates for low-speed urban streets.

2. Vertical Alignment. Even though the profile may satisfy all design controls, the use of minimum criteria may appear forced and angular. Therefore, the designer should use higher values to produce a smoother, more aesthetically pleasing alignment. Note that curves that are too flat may produce flat areas that may cause drainage problems. For further guidance, see Chapter 6 "Vertical Alignment."

3. Horizontal and Vertical Combinations. Consider the relationship between horizontal and vertical alignments simultaneously to obtain a desirable condition. Section 6.2.2 discusses this relationship in detail and its effect on aesthetics and safety.

4. Minimum Grades. Desirably, the longitudinal grade should be 0.5 percent or greater. For curbed facilities and bridges, it is necessary to provide a minimum longitudinal grade of 0.3 percent to facilitate drainage. For curbed sections, ensure curb profiles provide positive drainage. For uncurbed facilities, a minimum longitudinal grade of 0.0 percent may be considered if adequate cross slopes are provided. Ensure superelevation transitions are not developed in areas with 0.0 percent grades. Special ditch grades may be necessary to ensure proper drainage.

5. Climbing Lanes. Section 6.4 discusses the warrants and design criteria for climbing lanes. For most arterials, climbing lanes are generally only warranted on rural two-lane arterials.

16.2.6 Cross Section Elements

The following sections summarize cross section criteria for arterials. For additional information concerning cross sections, the designer should review Chapter 7 "Cross Section Elements."

16.2.6.1 Typical Sections

The following figures present typical sections for rural and urban arterials:

- Figure 16.2-A – Typical Rural Two-Lane Arterial
- Figure 16.2-B – Typical Rural Four-Lane Divided Arterial
- Figure 16.2-C – Typical Urban Four-Lane Divided Arterial
- Figure 16.2-D – Typical Urban Five-Lane Arterial (TWLTL) with Shoulders
- Figure 16.2-E – Typical Urban Five-Lane Arterial (TWLTL) with Curb and Gutter

16.2.6.2 Travel Lane and Shoulder Widths

Travel lane widths should be 12 feet. Provide a 10-foot shoulder, which includes a minimum paved width of 2 feet. In constrained urban areas with curb and gutter, the shoulder width may be just the 2-foot curb and gutter width. Where bicycles are to be accommodated on the shoulder, the designer should provide a minimum paved shoulder width of 4 feet. Low speed facilities with curb and gutter sections do not require a shoulder. For high speed facilities with curb and gutter sections, place the curb and gutter on the outside of the shoulder.

16.2.6.3 Cross Slopes

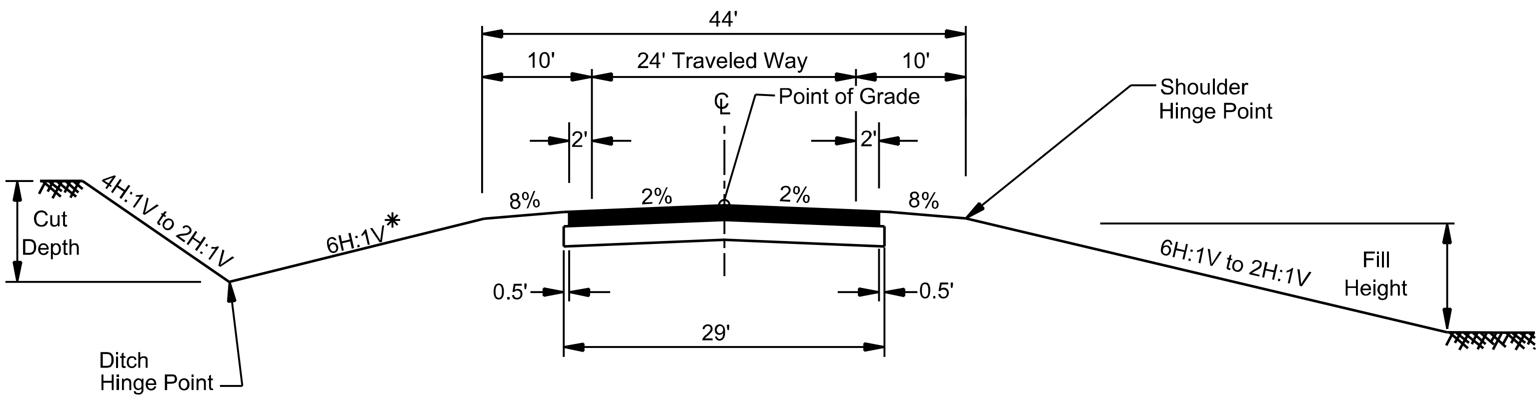
Use a cross slope of 2.00 percent for up to two lanes plus one half the width of the flush median or TWLTL. Travel lanes beyond the second lane on one side of the crown should have a cross slope of 2.50 percent. Crown the pavement at the center of the TWLTL and use a cross slope of 2.00 percent away from the centerline for all lanes on three- and five-lane highways. For a seven-lane section, use a cross slope of 2.50 percent for the outside lanes. If a roadway profile grade is less than 2.00 percent, the designer may consider using a cross slope of 2.50 percent for the outside lane to improve drainage. See Section 7.2.3.3.

For paved shoulders greater than 4 feet, provide a shoulder cross slope of 4.00 percent. For paved shoulders less than or equal to 4 feet, the cross slope should match the adjacent travel lane slope. For earth shoulders, provide a shoulder cross slope of 8.00 percent.

For cross slopes on bridges, see the *SCDOT Bridge Design Manual.*

16.2.6.4 Auxiliary Lanes

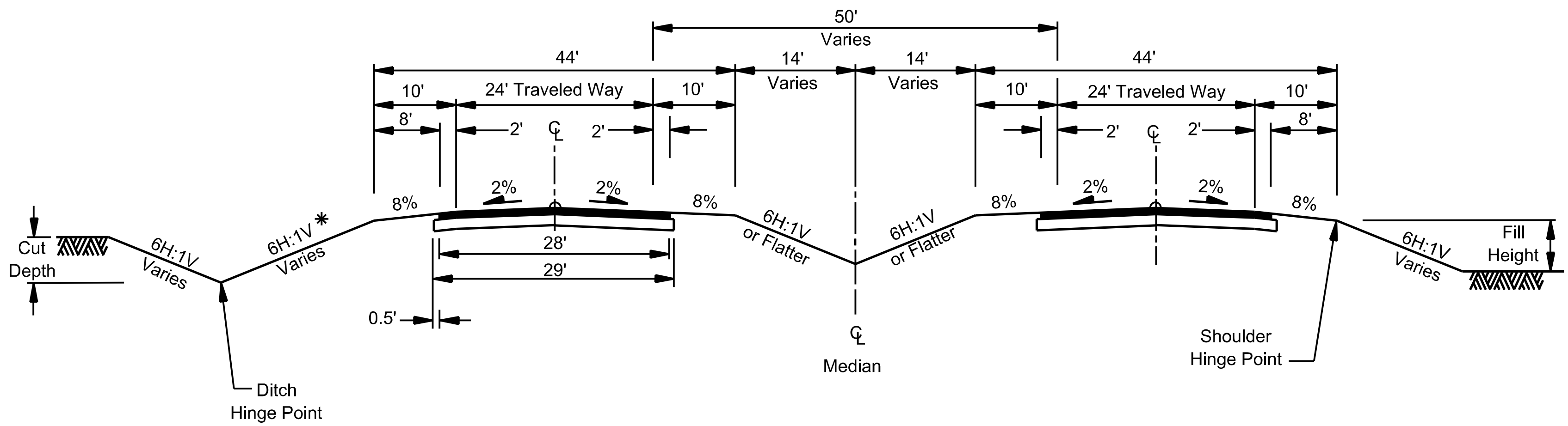
Auxiliary lanes (e.g., passing lanes, parking lanes, turn lanes) are lanes beyond the through travel lanes intended for use by vehicular traffic for specific functions. Desirably, auxiliary lanes will have the same width and cross slope as the adjacent through lanes, although in many cases a lesser width may be appropriate. The geometric design tables in Section 16.3 present lane and shoulder widths for auxiliary lanes.



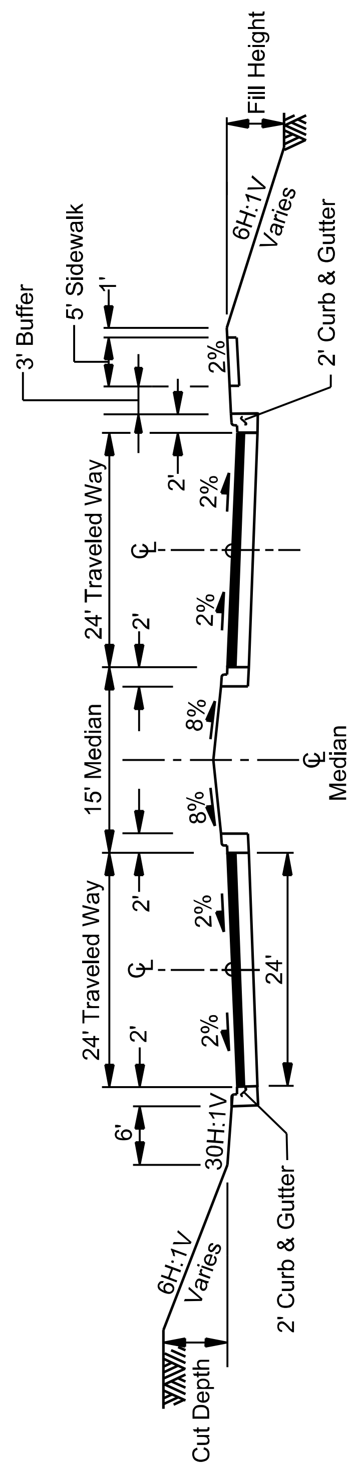
*This slope may vary between a minimum slope of 12H:1V to a maximum slope of 4H:1V. Where a deeper ditch than provided by a 4H:1V slope is necessary for drainage purposes, continue the 4H:1V slope until the necessary depth has been obtained. This will place the ditch further away from the roadway. Provide a separate profile for special ditch grades.

TYPICAL RURAL TWO-LANE ARTERIAL
Figure 16.2-A

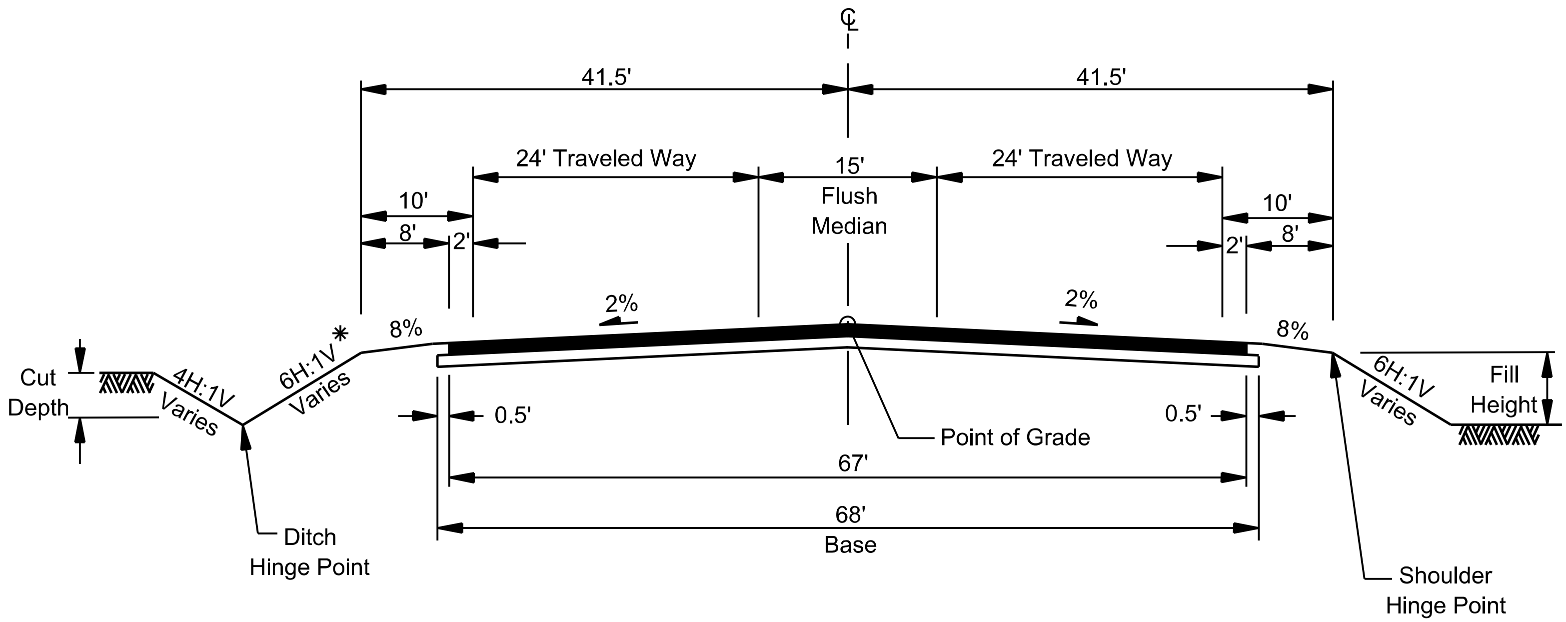
TYPICAL RURAL FOUR-LANE DIVIDED ARTERIAL
Figure 16.2-B



*This slope may vary between a minimum slope of 12H:1V to a maximum slope of 4H:1V. Where a deeper ditch than provided by a 4H:1V slope is necessary for drainage purposes, continue the 4H:1V slope until the necessary depth has been obtained. This will place the ditch further away from the roadway. Provide a separate profile for special ditch grades.

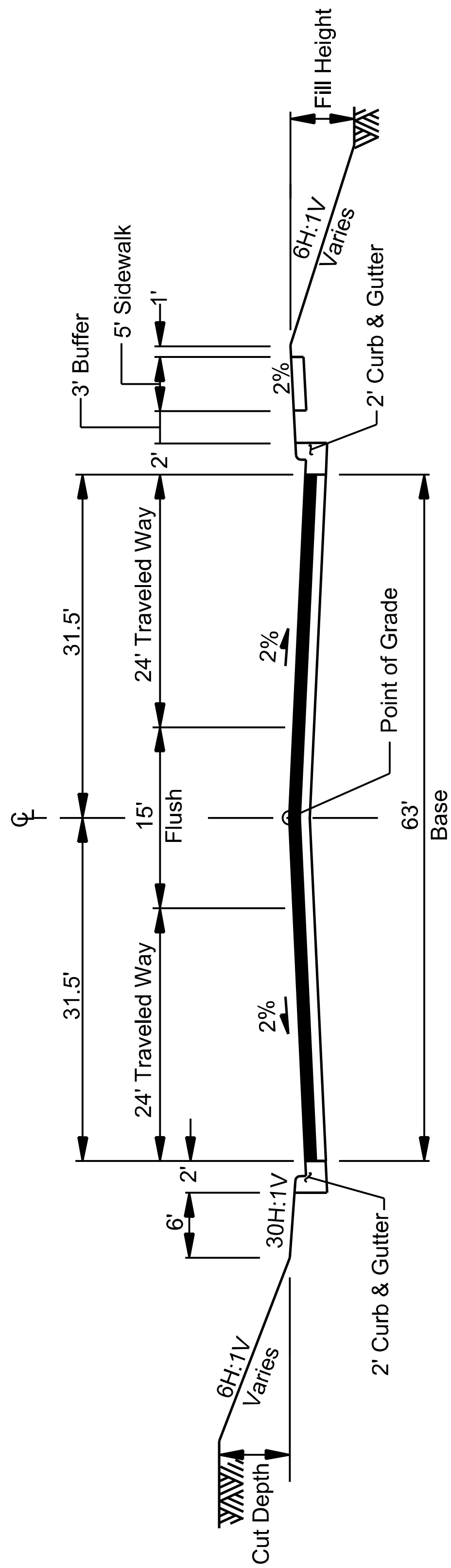


TYPICAL URBAN FOUR-LANE DIVIDED ARTERIAL
Figure 16.2-C



*This slope may vary between a minimum slope of 12H:1V to a maximum slope of 4H:1V. Where a deeper ditch than provided by a 4H:1V slope is necessary for drainage purposes, continue the 4H:1V slope until the necessary depth has been obtained. This will place the ditch further away from the roadway. Provide a separate profile for special ditch grades.

TYPICAL URBAN FIVE-LANE ARTERIAL (TWLTL) WITH SHOULDERS
Figure 16.2-D



TYPICAL URBAN FIVE-LANE ARTERIAL (TWLTL) WITH CURB AND GUTTER
Figure 16.2-E

16.2.6.5 Medians

A median should be considered on many multilane arterials. The principal functions of a median are to provide a separation from opposing traffic, to provide for turning movements, to manage access, to provide pedestrian refuge and to provide width for future lanes. Medians on arterials may be one of the following median types:

1. Flush Medians. Flush medians provide an area for left-turn movements and permit direct access to adjoining properties. This allows for numerous unrestricted conflict points. The flush median may serve as refuge for disabled vehicles and serve as a temporary lane for emergency vehicles. The two-way, left-turn lane (TWLTL) is considered a flush median. Desirably, the roadway cross section with a flush median will allow development of a TWLTL, where appropriate.

2. Raised Medians. Raised medians restrict left-turn movements to select locations, which allows for better access management. This median may provide a refuge area for pedestrians and an open space for aesthetic considerations.

3. Depressed Medians. Although additional right of way is required, depressed medians provide wider separation between opposing flows and greater drainage capabilities. Wide depressed medians may provide for future widening.

For guidance on medians and TWLTL, see Chapter 7 “Cross Section Elements.”

16.2.6.6 Transitions

The designer needs to give careful consideration to the design of transitions from multilane facilities to two-lane facilities; see Figure 16.2-F. These are complex decision-making areas for a driver, who may not be expecting the lane reduction.

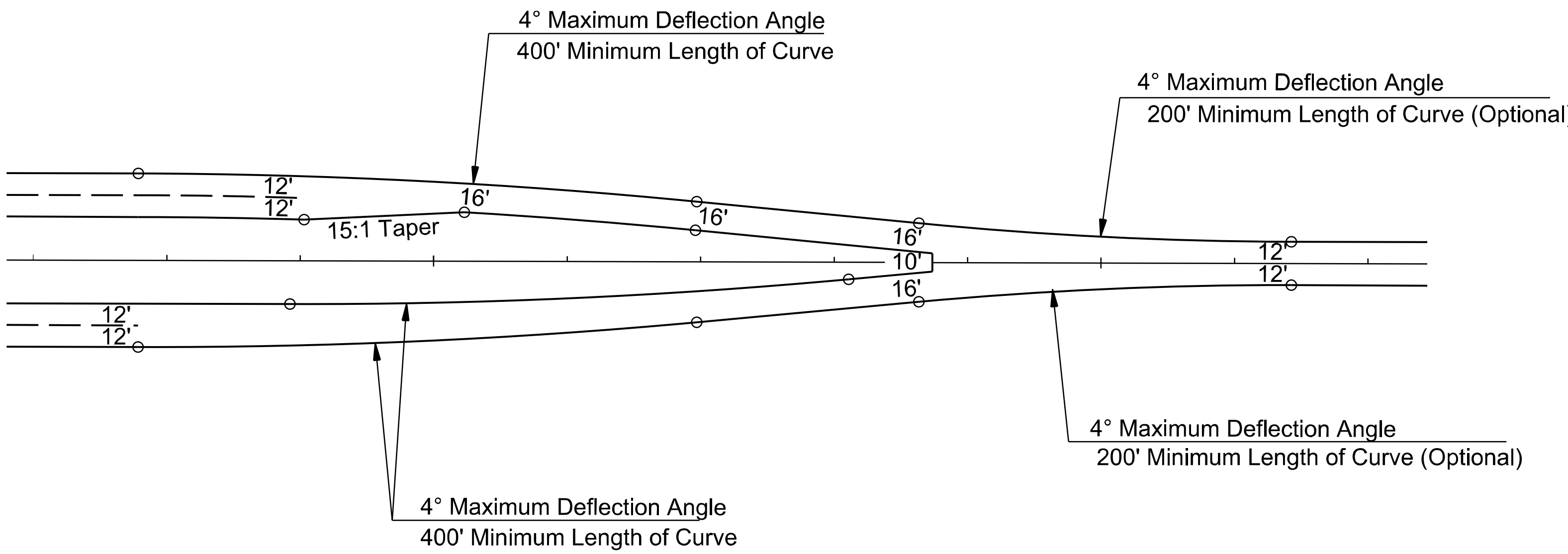
The horizontal alignment for permanent and temporary transitions should follow the criteria presented in Chapter 5 “Horizontal Alignment.” Desirably, design all temporary connections as new facilities. This includes, but is not limited to, superelevation, transition lengths, reverse curves and the tangent length between curves.

The designer should provide decision sight distance to and throughout the transition area. To achieve this objective, the project termini may need to be adjusted.

16.2.6.7 Frontage Roads

Arterial highways are generally characterized by their ability to quickly move relatively large volumes of traffic, but often with restricted capacity to serve abutting properties. A frontage road may be proposed in conjunction with an arterial to provide access to abutting properties. Where frontage roads are proposed in conjunction with a multilane arterial, provide a contiguous, but independent right of way adjacent to the mainline right of way for the frontage road. In certain instances, it may be advantageous to provide frontage roads that run parallel to the arterial, but have a wider separation resulting in two independent rights of way.

LANE TRANSITION DESIGN ON TANGENT SECTION FROM FOUR TO TWO LANES



Note: Lane Drop Occurs Prior to Lane Transition Taper (See MUTCD)

Figure 16.2-F

Where the profile grade of the arterial passes through major cuts and fills, the grade of the frontage road typically conforms to the existing ground line. The difference in elevation between the two adjacent facilities is provided for in the outer separations by earth slopes or possibly retaining walls. See Section 17.5.3 for additional information on frontage roads.

For rural multilane principal arterials, the frontage road should be designed using the rural two-lane arterial criteria, see Figure 16.3-A. For urban arterials, frontage roads may be designed as a two-lane urban arterial (Figure 16.3-C), collector (Chapter 15 "Collector Roads and Streets), or local street (Chapter 14 "Local Roads and Streets").

16.2.6.8 Right of Way

Providing right-of-way widths that accommodate construction, drainage and proper maintenance of a collector is an important part of the overall design. Wider right of way allows for gentler side slopes, which results in reduced crash severity potential and easier maintenance operations. Right of way is typically configured to accommodate all proposed cross section elements throughout the project (e.g., travel lanes, shoulders, medians, parking lanes, bike lanes, sidewalks, ditches, outer slopes). If a long-range plan identifies a future widening, give consideration to accommodating a future proposed cross section. A uniform right-of-way width is preferred; however, do not base the width on the critical point of the project. A critical point may occur where the side slopes extend beyond the normal right of way, for clear areas at the bottom of traversable slopes, for wider clear areas on the outside of curves, where greater sight distance is desirable, at intersections and junctions with other roads, at railroad-roadway grade crossings, for environmental considerations and for maintenance access.

16.2.7 Alternatives to Widening on Two-Lane Facilities

For rural two-lane arterials that are not candidates for widening to four-lane facilities, but are experiencing operational and safety problems or site-specific reductions in LOS, the designer should consider implementing one or more of the following improvements:

1. Realignment. Passing sight distance for two-lane rural highways is critical to the safe operation and capacity of highways. The designer should consider realignment of the arterial to improve horizontal and/or vertical alignment. These improvements inherently increase the passing sight distance, thereby, increasing safety and capacity. Minimum passing sight distances for two-lane highways are discussed in Section 4.2.

2. Medians. Medians can improve traffic flow and provide safety and operation benefits. Providing two-way, left-turn lanes (TWLTL) are viable alternatives where the number of left-turning vehicles is significant. Section 7.4 provides the guidelines and criteria for TWLTL. The use of the TWLTL for left-turning vehicles is typically not provided where design speed limits exceed 45 miles per hour. Therefore, their applicability to rural highways is usually near suburban areas or for roads passing through small towns. This alternative eliminates the possibility for the passing maneuver and does not increase through traffic capacity for the roadway segment.

3. Intersection Treatments. Depending on the access demands for a particular two-lane facility, intersections can be a critical part of a facility's design. The use of left-turn lanes

and bypass lanes to facilitate the movement and enhance the safety of through traffic at intersections is a cost-effective approach for upgrading two-lane highways. The designer should conduct detailed analyses of intersections in accordance with procedures in the *Highway Capacity Manual*. When modifying intersections, consider the following:

- design vehicle,
- signal warrants,
- sight distance,
- crash analyses,
- turning movements/lane warrants,
- intersection alignment,
- rights of way requirements,
- LOS analysis, and
- economic factors.

For additional guidance, see Chapter 9 “Intersections.”

4. Climbing Lanes. In areas with steep grades, reduced truck speeds may significantly affect the facility’s capacity and safety. However, truck-climbing lanes can effectively increase capacity and safety. Warrants and design criteria for truck-climbing lanes are discussed in Section 6.4.

5. Passing Lanes. Passing lanes, other than truck-climbing lanes, may be warranted on two-lane facilities where passing opportunities are not adequate or when an engineering study, operational experience and a capacity analysis concludes that there is a critical need. Section 7.2.6 discusses passing lane designs.

16.2.8 Roadside Safety

The designer should provide adequate horizontal clearance between the traveled way and roadside obstructions on arterials. The designer should provide roadside clear zones as discussed in the AASHTO *Roadside Design Guide*.

16.3 TABLES OF DESIGN CRITERIA

The geometric design tables in this section present the Department's design and alignment criteria for rural and urban arterial projects. The designer should consider the following when using these tables:

1. Applicability. Note that some of the cross-section elements included in the tables (e.g., TWLTL) are not automatically warranted in the project design. The values in the figures only apply after the decision has been made to include the design element in the highway cross section.

2. Manual Section References. These tables are intended to provide a concise listing of design values for easy use. However, the designer should review the *Highway Design Manual* section references for more information on the design elements.

3. Footnotes. The figures include many footnotes, which are identified by a number in parentheses (e.g., **(3)**). The information in the footnotes is critical to the proper use of the design tables.

The following design tables are provided for arterials:

- Figure 16.3-A — "Geometric Design Criteria for Rural Two-Lane Arterials (New Construction/Reconstruction)"
- Figure 16.3-B — "Geometric Design Criteria for Rural Multilane Arterials (New Construction/Reconstruction)"
- Figure 16.3-C — "Alignment Criteria for Rural Arterials (New Construction/Reconstruction)"
- Figure 16.3-D — "Geometric Design Criteria for Urban Two-Lane Arterials (New Construction/Reconstruction)"
- Figure 16.3-E — "Geometric Design Criteria for Urban Multilane Arterials (New Construction/Reconstruction)"
- Figure 16.3-F — "Alignment Criteria for Urban Arterials (New Construction/Reconstruction)"

Design Element				Manual Section	Rural
Design Controls	Design Forecast Year			16.2.1	20 Years
	Design Speed	Level		16.2.3	Minimum: 60 mph
		Rolling			Minimum: 50 mph
		Mountainous			Minimum: 40 mph
	Access Control			3.8	Control by Regulation
	Level of Service			3.6.4	Level/Rolling: B Mountainous: C
Cross Section Elements	Travel Lane Width **(1)**			16.2.6	12 ft
	Shoulder	Total Width (**2)**		16.2.6	10 ft
		Paved Width			2 ft
	Auxiliary Lanes	Lane Width		16.2.6	12 ft
		Shoulder Width	Total		10 ft
			Paved		2 ft
	Cross Slope	Travel Lane		16.2.6	2.00%
		Auxiliary Lane			2.00%
		Shoulder	Paved **(3)**		2.00%
			Unpaved		8.00%
	TWLTL			16.2.6	15 ft
	Right of Way Width			16.2.6	Project Specific
Roadway Slopes	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V to 4H:1V
			Ditch Type		V-Ditch
			Back Slope		4H:1V to 2H:1V
			Rock Cut		0.25H:1V
		Fill Section	0 ft – 5 ft	7.3.2	6H:1V
			5 ft – 10 ft		4H:1V
			> 10 ft		2H:1V
	Clear Zone				**(4)**

GEOMETRIC DESIGN CRITERIA FOR RURAL TWO-LANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-A
(Continued on next page)

Design Element			Manual Section	Rural
Structures	New Bridges	Structural Capacity	7.5.1	HL-93
		Clear Roadway Width **(5)**		44 ft
	Existing Bridges to Remain in Place	Structural Capacity	7.5.1	**(6)**
		Clear Roadway Width **(5)**		44 ft
	Vertical Clearance (Arterial Under) **(7a)**	New and Replaced Overpassing Bridges **(7b)**	6.6	17 ft – 0 in
		Existing Overpassing Bridges		16 ft – 0 in
		Pedestrian Bridges		18 ft – 0 in
		Overhead Signs		17 ft – 6 in
		Overhead Utilities		Coordinate with Utility Office
	Clearance (Arterial Over)	Railroads	6.6	23 ft – 0 in
		Underpass Width	7.5.2	Traveled Way plus Clear Zone
	Vertical Clearance (Over Water)	Navigable Water	6.6	See Environmental Services Office
		Major Lakes & Reservoirs (with boat traffic)		8 ft – 0 in above the high water mark
		Rivers		2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.
		Tidal Waters		2 ft above the 10-year high water elevation including wave height.

Footnotes

(1) Travel Lane Width. An existing 22-foot traveled way may be retained where the alignment is satisfactory and there is no crash pattern suggesting the need for widening.

(2) Shoulder (Total Width). Where guardrail is required, increase the shoulder width an additional 3.5 feet.

(3) Shoulder Cross Slope. For paved shoulders wider than 4 feet, provide a 4.00 percent shoulder cross slope.

(4) Clear Zone. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(5) Bridge Widths. Clear roadway bridge widths are measured from face to face of parapets or rails. Bridge widths are normally defined as the sum of the approach traveled way width plus total width for both shoulders. See Section 7.5.1.1 for further guidance.

(6) Existing Bridges to Remain in Place. Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(7) Vertical Clearance (Arterial Under).

(a) The clearance must be available over the traveled way, shoulders and any future widening identified in a long-range plan.

(b) Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR RURAL TWO-LANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-A
(Continued)

Design Element				Manual Section	Rural
Design Controls	Design Forecast Year			16.2.1	20 Years
	Design Speed	Level		16.2.3	Minimum: 60 mph
		Rolling			Minimum: 50 mph
		Mountainous			Minimum: 40 mph
	Access Control			3.8	Control by Regulation
	Level of Service			3.6.4	Level/Rolling: B Mountainous: C
Cross Section Elements	Travel Lane Width **(1)**			16.2.6	12 ft
	Shoulder Width	Right	Total **(2)**	16.2.6	10 ft
			Paved		2 ft
		Left	Total **(2)**		10 ft
			Paved		2 ft
	Auxiliary Lanes	Lane Width		16.2.6	12 ft
		Shoulder Width	Total		10 ft
			Paved		2 ft
	Cross Slope	Travel Lane		16.2.6	2.00%
		Auxiliary Lane			2.00%
		Shoulder	Paved **(3)**		2.00%
			Unpaved		8.00%
	Median Width	Depressed		16.2.6	48 ft
	Right of Way Width			16.2.6	Project Specific
Roadway Slopes	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V to 4H:1V
			Ditch Type		V-Ditch
			Back Slope		6H:1V to 2H:1V
			Rock Cut		0.25H:1V
		Fill Section	0 ft – 5 ft	7.3.2	6H:1V
			5 ft – 10 ft		4H:1V
			> 10 ft		2H:1V
	Clear Zone				**(4)**

GEOMETRIC DESIGN CRITERIA FOR RURAL MULTILANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-B

(Continued on next page)

<table>
<tr><th colspan="3">Design Element</th><th>Manual Section</th><th>Rural</th></tr>
<tr><td rowspan="17">Structures</td><td rowspan="2">New Bridges</td><td>Structural Capacity</td><td rowspan="2">7.5.1</td><td>HL-93</td></tr>
<tr><td>Clear Roadway Width (5)</td><td>44 ft</td></tr>
<tr><td rowspan="2">Existing Bridges to Remain in Place</td><td>Structural Capacity</td><td rowspan="2">7.5.1</td><td>(6)</td></tr>
<tr><td>Clear Roadway Width (5)</td><td>44 ft</td></tr>
<tr><td rowspan="5">Vertical Clearance (Arterial Under) (7a)</td><td>New and Replaced Overpassing Bridges (7b)</td><td rowspan="5">6.6</td><td>17 ft – 0 in</td></tr>
<tr><td>Existing Overpassing Bridges</td><td>16 ft – 0 in</td></tr>
<tr><td>Pedestrian Bridges</td><td>18 ft – 0 in</td></tr>
<tr><td>Overhead Signs</td><td>17 ft – 6 in</td></tr>
<tr><td>Overhead Utilities</td><td>Coordinate with Utility Office</td></tr>
<tr><td rowspan="2">Vertical Clearance (Arterial Over)</td><td>Railroads</td><td>6.6</td><td>23 ft – 0 in</td></tr>
<tr><td>Underpass Width</td><td>7.5.2</td><td>Traveled Way plus Clear Zone</td></tr>
<tr><td rowspan="4">Vertical Clearance (Over Water)</td><td>Navigable Water</td><td rowspan="4">6.6</td><td>See Environmental Services Office</td></tr>
<tr><td>Major Lakes & Reservoirs (with boat traffic)</td><td>8 ft – 0 in above the high water mark</td></tr>
<tr><td>Rivers</td><td>2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.</td></tr>
<tr><td>Tidal Waters</td><td>2 ft above the 10-year high water elevation including wave height.</td></tr>
</table>

Footnotes

(1) Travel Lane Width. On reconstructed arterials, an existing 22-foot traveled way may be retained where the alignment is satisfactory and there is no crash pattern suggesting the need for widening.

(2) Shoulder (Total Width). Where guardrail is required, increase the shoulder width an additional 3.5 feet.

(3) Shoulder Cross Slope. For paved shoulders wider than 4 feet, provide a 4.00 percent shoulder cross slope.

(4) Clear Zone. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(5) Bridge Widths. Clear roadway bridge widths are measured from face to face of parapets or rails. Bridge widths are normally defined as the sum of the approach traveled way width plus the widths for the left and right shoulders. See Section 7.5.1.1 for further guidance.

(6) Existing Bridges to Remain in Place. Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(7) Vertical Clearance (Arterial Under).

(a) The clearance must be available over the traveled way, shoulders and any future widening identified in a long-range plan.

(b) Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR RURAL MULTILANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-B
(Continued)

Design Element		Manual Section	Design Speed						
			40 mph	**50 mph**	**55 mph**	**60 mph**	**65 mph**	**70 mph**	**75 mph**
Stopping Sight Distance **(1)**		4.1	305 ft	425 ft	495 ft	570 ft	645 ft	730 ft	820 ft
Passing Sight Distance		4.2	600 ft	800 ft	900 ft	1000 ft	1100 ft	1200 ft	1300 ft
Decision Sight Distance **(2)**		4.3	600 ft	750 ft	865 ft	990 ft	1050 ft	1105 ft	1180 ft
Intersection Sight Distance **(3)**		4.4	445 ft	555 ft	610 ft	665 ft	720 ft	775 ft	830 ft
Minimum Radii	e_{max} = 8%	5.2		758 ft	960 ft	1200 ft	1480 ft	1810 ft	2210 ft
	e_{max} = 6%		485 ft	833 ft					
Superelevation Rate **(4)**		5.3	6%	6% or 8%	8%	8%	8%	8%	8%
Horizontal Sight Line Offset **(5)**		5.4	24 ft	30 ft	32 ft	34 ft	35 ft	37 ft	38 ft
Vertical Curvature (K-Values) **(6)**	Crest	6.5	44	84	114	151	193	247	312
	Sag		64	96	115	136	157	181	206
Maximum Grade	Level	6.3.1	5%	4%	4%	3%	3%	3%	3%
	Rolling		6%	5%	5%	4%	4%	4%	4%
	Mountainous		8%	7%	6%	6%	5%	5%	5%
Minimum Grade **(7)**		6.3.2	0.5%						

Footnotes

(1) Stopping Sight Distance. Table values are for passenger cars on level grade.

(2) Decision Sight Distance. Table values are for speed/path/direction change on rural road, Avoidance Maneuver C. See Section 4.3 for other maneuvers.

(3) Intersection Sight Distance. Table values are for passenger cars for assumed conditions described in Figure 4.4-C. See Section 4.4 for truck values.

(4) Superelevation Rate. See Section 5.3 for superelevation rates based on e_{max}, design speed and radii of horizontal curves. For horizontal curves to remain in place, an e_{max} of 8 percent may be considered to remain in place. Where a crossroad intersection lies within the limits of a mainline horizontal curve, see Figure 5.3-A for the maximum superelevation rates allowed on the mainline curve.

(5) Horizontal Sight Line Offset. Table values provide the necessary middle ordinate assuming the design speed, stopping sight distance and minimum radii based on an e_{max} = 6 percent for the 40 miles per hour design speed or e_{max} = 8 percent for design speeds of 50 to 75 miles per hour.

(6) Vertical Curvature (K-Value). K-values are based on the level stopping sight distances.

(7) Minimum Grade. Longitudinal gradients of 0.0 percent may be acceptable on some pavements that have cross slopes that have adequate drainage. Ensure superelevation transitions are not developed in areas with 0.0 percent grade. Special ditch grades may be necessary to ensure proper project runoff management.

ALIGNMENT CRITERIA FOR RURAL ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-C

Design Element				Manual Section	Urban
Design Controls	Design Forecast Year			16.2.1	20 Years
	Design Speed			16.2.3	Minimum: 30 mph
	Access Control			3.8	Limited/Control by Regulation
	Level of Service			3.6.4	Desirable: C
Cross Section Elements	Travel Lane Width **(1)**			16.2.6	12 ft
	Shoulder Width	Total		16.2.6	10 ft or Curb and Gutter
		Paved			2 ft or Curb and Gutter
	Auxiliary Lanes	Lane Width		16.2.6	12 ft
		Shoulder Width	Total		10 ft or Curb and Gutter
			Paved		2 ft or Curb and Gutter
	Parking Lane Width **(2)**			7.2.7	12 ft
	Cross Slope	Travel Lane		16.2.6	2.00%
		Auxiliary Lane			2.00%
		Shoulder	Paved **(3)**		2.00%
			Unpaved		8.00%
	Bicycle	Lane Width **(4)**		11.11	4 ft
		Shared Roadway Width			14 ft Outside Travel Lane
	Curb and Gutter	Type **(5)**		7.2.8	Vertical or Sloping
		Width			2 ft
	Sidewalk Width			7.3.3	5 ft
	Median Width	TWLTL		16.2.6	15 ft
		Raised			Desirable: 12 ft Minimum: 4 ft
	Right of Way Width			16.2.6	Project Specific
Roadway Slopes	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V to 4H:1V
			Ditch Type		V-Ditch
			Back Slope **(6)**		4H:1V to 2H:1V
		Fill Section	0 ft – 5 ft	7.3.2	6H:1V
			5 ft – 10 ft		4H:1V
			> 10 ft		2H:1V
	Flush/TWLTL Slopes			7.3.2	2.00%
	Clear Zone				**(7)**

GEOMETRIC DESIGN CRITERIA FOR URBAN TWO-LANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-D
(Continued on next page)

Design Element			Manual Section	Urban
Structures	New Bridges	Structural Capacity	7.5.1	HL-93
		Clear Roadway Width		**(8)**
	Existing Bridges to Remain in Place	Structural Capacity	7.5.1	**(9)**
		Clear Roadway Width		**(8)**
	Vertical Clearance (Arterial Under) **(10a)**	New and Replaced Overpassing Bridges **(10b)**	6.6	17 ft – 0 in
		Existing Overpassing Bridges	6.6	16 ft – 0 in
		Pedestrian Bridges	6.6	18 ft – 0 in
		Overhead Signs	6.6	17 ft – 6 in
		Overhead Utilities	6.6	Coordinate with Utility Office
	Vertical Clearance (Arterial Over)	Railroads	6.6	23 ft – 0 in
		Underpass Width	7.5.2	Traveled Way plus Clear Zone
	Vertical Clearance (Over Water)	Navigable Water	6.6	See Environmental Services Office
		Major Lakes & Reservoirs (with boat traffic)		8 ft – 0 in above the high water mark
		Rivers		2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.
		Tidal Waters		2 ft above the 10-year high water elevation including wave height.

GEOMETRIC DESIGN CRITERIA FOR URBAN TWO-LANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-D
(Continued on next page)

Footnotes for Figure 16.3-D

(1) Travel Lane Width. In CBDs, an 11-foot traveling lane may be used if the truck volumes are less than or equal to 5 percent.

(2) Parking Lane Width. A parking lane width as narrow as 10 feet may be acceptable.

(3) Shoulder Cross Slope. For paved shoulders wider than 4 feet, provide a 4.00 percent shoulder cross slope.

(4) Bicycle (Lane Width). For design speeds greater than 45 miles per hour, increase the bike lane width in accordance with AASHTO *Guide for the Development of Bicycle Facilities*.

(5) Curb and Gutter (Type). If curb and gutter is used on streets with design speeds greater than 45 miles per hour, place the curb and gutter outside of the shoulder and use a sloping curb.

(6) Side Slopes. Generally on curb and gutter sections, provide a maximum slope of 2H:1V.

(7) Clear Zone. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(8) Bridge Widths. Clear roadway bridge widths are measured from face to face of parapets or rails. Bridge widths are normally defined as the sum of the approach traveled way width, both shoulders and median width, where applicable. For curbed sections, the clear roadway width will be the curb-to-curb width plus the sidewalk width on one or both sides. See Section 7.5.1.1 for further guidance.

(9) Existing Bridges to Remain in Place. Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(10) Vertical Clearance (Arterial Under).

a. The clearance must be available over the traveled way, shoulders and any future widening identified in a long-range plan.

b. Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR URBAN TWO-LANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-D
(Continued)

Design Element				Manual Section	Urban
Design Controls	Design Forecast Year			16.2.1	20 Years
	Design Speed			16.2.3	Minimum: 30 mph
	Access Control			3.8	Limited/Control by Regulation
	Level of Service			3.6.4	Desirable: C
Cross Section Elements	Travel Lane Width **(1)**			16.2.6	12 ft
	Shoulder Width	Right	Total	16.2.6	10 ft or Curb and Gutter
			Paved		2 ft or Curb and Gutter
		Left	Total		10 ft or Curb and Gutter
			Paved		2 ft or Curb and Gutter
	Auxiliary Lanes	Lane Width		16.2.6	12 ft
		Shoulder Width	Total		10 ft or Curb and Gutter
			Paved		2 ft or Curb and Gutter
	Parking Lane Width **(2)**			7.2.7	12 ft
	Cross Slope	Travel Lane		16.2.6	2.00%
		Auxiliary Lane			2.00%
		Shoulder	Paved **(3)**		2.00%
			Unpaved		8.00%
	Bicycle	Lane Width **(4)**		11.11	4 ft
		Shared Roadway Width			14 ft Outside Travel Lane
	Curb and Gutter	Type **(5)**		7.2.8	Vertical or Sloping
		Width			2 ft
	Sidewalk Width			7.3.3	5 ft
	Median Width	TWLTL		16.2.6	15 ft
		Raised			Desirable: 12 ft Minimum: 4 ft
	Right of Way Width			16.2.6	Project Specific
Roadway Slopes	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V to 4H:1V
			Ditch Type		V-Ditch
			Back Slope **(6)**		4H:1V to 2H:1V
		Fill Section	0 ft – 5 ft	7.3.2	6H:1V
			5 ft – 10 ft		4H:1V
			> 10 ft		2H:1V
	Median Slopes	Depressed		7.3.2	6H:1V
		Flush/TWLTL			2.00%
	Clear Zone				**(7)**

GEOMETRIC DESIGN CRITERIA FOR URBAN MULTILANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-E
(Continued on next page)

Design Element			Manual Section	Urban
Structures	New Bridges	Structural Capacity	7.6.1	HL-93
		Clear Roadway Width		**(8)**
	Existing Bridges Remain in Place	Structural Capacity	7.6.1	**(9)**
		Clear Roadway Width		**(8)**
	Vertical Clearance (Arterial Under) **(**10**a)**	New and Replaced Overpassing Bridges **(10b)**	6.6	17 ft – 0 in
		Existing Bridges	6.6	16 ft – 0 in
		Pedestrian Bridges	6.6	18 ft – 0 in
		Overhead Signs	6.6	17 ft – 6 in
		Overhead Utilities	6.6	Coordinate with Utility Office
	Vertical Clearance (Arterial Over)	Railroads	6.6	23 ft – 0 in
		Underpass Width	7.6.2	Traveled Way plus Clear Zone
	Vertical Clearance (Over Water)	Navigable Water	6.6	See Environmental Services Office
		Major Lakes & Reservoirs (with boat traffic)		8 ft – 0 in above the high water mark
		Rivers		2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.
		Tidal Waters		2 ft above the 10-year high water elevation including wave height.

GEOMETRIC DESIGN CRITERIA FOR URBAN MULTILANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-E

(Continued on next page)

Footnotes for Figure 16.3-E

(1) Travel Lane Width. In CBDs, an 11-foot traveling lane may be used if the truck volumes are less than or equal to 5 percent.

(2) Parking Lane Width. A parking lane width as narrow as 10 feet may be acceptable.

(3) Shoulder Cross Slope. For paved shoulders wider than 4 feet, provide a 4.00 percent shoulder cross slope.

(4) Bicycle (Lane Width). For design speeds greater than 45 miles per hour, increase the bike lane width in accordance with AASHTO *Guide for the Development of Bicycle Facilities*.

(5) Curb and Gutter (Type). If curb and gutter is used on streets with design speeds greater than 45 miles per hour, place the curb and gutter outside of the shoulder and use a sloping curb.

(6) Side Slopes. Generally on curb and gutter sections, provide a maximum slope of 2H:1V.

(7) Clear Zone. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(8) Bridge Widths. Clear roadway bridge widths are measured from face to face of parapets or rails. Bridge widths are normally defined as the sum of the approach traveled way width, right and left shoulders and median width, where applicable. For curbed sections, the clear roadway width will be the curb-to-curb width plus the sidewalk width on one or both sides. See Section 7.5.1.1 for further guidance.

(9) Existing Bridges to Remain in Place. Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(10) Vertical Clearance (Arterial Under).

(a) The clearance must be available over the traveled way, shoulders and any future widening identified in a long-range plan.

(b) Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR URBAN MULTILANE ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-E
(Continued)

Design Element		Manual Section	Design Speed						
			30 mph	**35 mph**	**40 mph**	**45 mph**	**50 mph**	**55 mph**	**60 mph**
Stopping Sight Distance **(1)**		4.1	200 ft	250 ft	305 ft	360 ft	425 ft	495 ft	570 ft
Decision Sight Distance **(2)**		4.3	490 ft	590 ft	690 ft	800 ft	910 ft	1030 ft	1150 ft
Intersection Sight Distance **(3)**		4.4	335 ft	390 ft	445 ft	500 ft	555 ft	610 ft	665 ft
Minimum Radii	e_{max} = 8%	5.2					758 ft	960 ft	1200 ft
	e_{max} = 6%		231 ft	340 ft	485 ft	643 ft	833 ft		
	e_{max} = 4%		250 ft	371 ft	533 ft	711 ft			
Superelevation Table **(4a)**		5.3	4% or 6% **(4b)**	4% or 6% **(4b)**	4% or 6% **(4b)**	4% or 6% **(4b)**	6% or 8%	8%	8%
Horizontal Sight Line Offset **(5)**		5.4	21 ft	23 ft	24 ft	25 ft	27 ft	32 ft	34 ft
Vertical Curvature (K-Values) **(6)**	Crest	6.5	19	29	44	61	84	114	151
	Sag		37	49	64	79	96	115	136
Maximum Grade	Level	6.3.1	8%	7%	7%	6%	6%	5%	5%
	Rolling		9%	8%	8%	7%	7%	6%	6%
	Mountainous		11%	10%	10%	9%	9%	8%	8%
Minimum Grade		6.3.2	Desirable: 0.5% Minimum: 0.3% (Curb and Gutter)						

Footnotes

(1) Stopping Sight Distance. Table values are for passenger cars on level grade.

(2) Decision Sight Distance. Table values are for stop on an urban road, Avoidance Maneuver B, as described in Figure 4.3-A.

(3) Intersection Sight Distance. Table values are for passenger cars for assumed conditions described in Figure 4.4-C. See Section 4.4 for truck values.

(4) Superelevation Rate.

(a) See Section 5.3 for superelevation rates based on e_{max}, design speed and radii of horizontal curves.
(b) The 6 percent superelevation rate should only be used on suburban arterials.

(5) Horizontal Sight Line Offset. Table values provide the necessary middle ordinate assuming the design speed, stopping sight distance and minimum radii based on an e_{max} = 6 percent for design speeds 30 to 50 miles per hour or e_{max} = 8 percent for design speeds of 55 and 60 miles per hour.

(6) Vertical Curvature (K-Value). K-values are based on the level stopping sight distances.

ALIGNMENT CRITERIA FOR URBAN ARTERIALS
(New Construction/Reconstruction)
Figure 16.3-F

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16.4 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. *Highway Safety Design and Operations Guide*, AASHTO, 1997.

3. *Roadside Design Guide*, AASHTO, 2012.

4. *Highway Capacity Manual*, Transportation Research Board, 2010.

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Chapter 17

FREEWAYS

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 17
FREEWAYS

17.1 GENERAL

Freeways/Interstates are functionally classified as principal arterials constructed with full control of access. Freeways are intended to provide high levels of safety and efficiency in moving high volumes of traffic at high speeds. The operational efficiency, capacity, safety and cost of the highway facility are largely dependent upon its design. Rural freeways connect roadway links between major cities, towns and urban areas. Similarly, urban freeways provide service for large volumes of traffic within and through urban areas. This chapter provides guidance in the design of rural and urban freeways/Interstates including specific design criteria, frontage roads, lane drops, justification for grade separations, access control along the freeway and safety. Additional information that is applicable to freeways is also included in the following chapters:

- Chapter 3 “Basic Design Controls” provides guidelines for access control along interchange crossroads. It also discusses the procedures for depicting control of access in the plans.
- Chapters 3 “Basic Design Controls,” 4 “Sight Distance,” 5 “Horizontal Alignment,” 6 “Vertical Alignment” and 7 “Cross Section Elements” provide guidance on geometric design elements applicable to freeways.
- Chapter 10 “Interchanges” discusses type, location and design of interchanges and ramps.

17.1.1 <u>Rural Versus Urban Freeways</u>

Rural and urban freeways are similar in that they are intended to provide safe, rapid and high-quality transportation facilities for motorists. However, it is important to consider the differences between rural and urban freeways when applying freeway design criteria to a project. The primary difference between rural and urban freeways is in the concept of operational freedom. Motorists on rural freeways expect more operational freedom and higher travel speeds than urban motorists. Rural freeways are normally constructed at ground level and right of way is typically less constrained.

Urban freeways are designed to carry large traffic volumes and have multiple lanes in each direction. Typically, urban freeways have two lanes in each direction, and can be designed for three or more lanes in each direction. Urban freeways take many forms (e.g., depressed, ground level, elevated embankment, elevated viaduct or a combination of these forms). In most instances, right-of-way restrictions require designers to evaluate various proposed alternatives to balance socio-economic, right of way, environmental and construction factors.

17.1.2 <u>Design Studies</u>

Freeway design considerations evolve around design year traffic volumes, design speed and level of service. These are the primary factors that either individually or collectively are instrumental in governing the selection of appropriate roadway geometric criteria and/or cross section elements. When developing a freeway alignment, the designer should first determine the location and types of interchanges then develop the freeway alignment between the interchanges. Other factors that may influence the freeway alignment include:

- the location of grade separations, including major river crossings;
- access control along the freeway and along interchange crossroads;
- topography;
- environmental restrictions; and
- property lines and right-of-way restrictions.

17.1.3 Project Scope of Work

17.1.3.1 New Construction

A freeway is considered new construction when a freeway is on new location and has a new alignment. The designer is required to meet all the criteria presented in Chapter 17 and the applicable criteria provided elsewhere in this *Manual*.

17.1.3.2 Reconstruction

Reconstruction of an existing freeway will typically include the addition of travel lanes, reconstruction of existing horizontal and vertical alignment, widening the roadway and flattening side slopes, but the freeway will remain essentially within the existing highway corridor. Because of the significant level of work for reconstruction, the design of the project will meet the criteria for new construction presented in Chapter 17 and the applicable criteria provided elsewhere in this *Manual*.

17.2 DESIGN ELEMENTS

17.2.1 Traffic

17.2.1.1 Traffic Volume

Traffic volumes are a major consideration in justifying highway facilities and in the establishment of preliminary geometric and cross section design characteristics. The designer should use the design year traffic volumes to determine the design elements of urban and rural freeways.

The traffic volumes used for design of Interstates should be the 30th highest hourly volume of the design year. This is the total traffic in both directions of travel is referred to as the design hourly volume (DHV).

17.2.1.2 Level of Service

The level of service (LOS) consideration in the design of freeways is determined using the *Highway Capacity Manual* or traffic modeling software. Because LOS is a measure of the freedom of movement and operational delays for traffic, it is appropriate to design freeways to operate at a high LOS.

Rural freeways should be designed to operate at a LOS B. LOS B is in the stable traffic flow range in which the motorist's freedom to select the desired operating speed is relatively unaffected and motorist's freedom of maneuvering is only slightly restricted. In rural mountainous terrain, it may be necessary to reduce the design to LOS C in which the ability to maneuver within the traffic stream becomes increasingly affected by the presence of other vehicles. Further discussion on the LOS design concept is included in Section 3.6.4.

For urban freeways, a LOS C is desirable, but in some cases it may not be economically feasible.

17.2.2 Design Speed

Freeways are intended to accommodate high speed traffic. For rural freeways, the design speed should be 70 miles per hour or higher. In mountainous terrain, rural freeway design speeds may be reduced to 55 miles per hour. For urban freeways, limited right of way, high construction costs and social or environmental concerns may suggest a lower design speed. Urban freeway design speeds should be at least 60 miles per hour to maintain an overall high quality, smooth-flowing facility. Decisions to use design speeds less than 60 miles per hour may be justified and supported with appropriate documentation.

17.2.3 Alignment

Designed for high volume and high speed operations, freeways should have smooth horizontal and vertical alignments. Proper combinations of curvature, tangents, grades, variable median widths and separate roadway elevations all combine to enhance safety and aesthetics of freeways. When designing freeway alignments, the designer should consider the following guidelines:

1. Horizontal Alignment. The following guidelines should be applied when laying out the horizontal alignment:

    - Use large radius curves.

    - Only use minimum radii where it is necessary due to restricted conditions.

    - Avoid alignments that require superelevation transitions on bridges or bridge approach slabs.

    - Consider sight distance restrictions from longitudinal barriers on the roadside and in the median.

2. Vertical Alignment. Even though the profile may satisfy all design controls, the use of minimum criteria may appear forced and angular. Therefore, the designer should use higher values to produce a smoother, more aesthetically pleasing alignment keeping in mind curves that are too flat will produce flat areas that may cause drainage problems.

3. Horizontal and Vertical Combinations. Consider the relationship between horizontal and vertical alignments simultaneously to obtain a desirable condition. Section 6.2.2 discusses this relationship in detail and its effect on aesthetics and safety.

4. Minimum Grades. Desirably, the longitudinal grade should be 0.5 percent or greater. For bridges, it is necessary to provide a minimum longitudinal grade of 0.3 percent to facilitate drainage. For uncurbed facilities, a minimum longitudinal grade of 0.0 percent may be considered if adequate cross slopes are provided. Ensure superelevation transitions are not developed in areas with 0.0 percent grades. Special ditch grades may be necessary to ensure proper drainage.

5. Freeway River Crossings. During the development of freeways, the alignment may need to cross major rivers, streams or bays. In selecting the location for a bridge site, consider the following guidelines:

    a. Crossing Angle. Where practical, cross the river at a right angle to minimize the length of the bridge.

    b. Bluffs. If a bluff exists adjacent to the river, try to locate one of the abutments on a bluff closest to the river. This will minimize the overall length of the bridge and, therefore, reduce the cost of the structure.

    c. River Bends. Avoid locating the bridge on a bend in the river. Locating a bridge on a bend may result in unnecessarily long spans and may increase the chance of ships and boats colliding with the main bridge piers.

    d. Freeway Alignment. Examine how the freeway alignment will tie into the ends of the bridge. Approach horizontal and vertical alignments can significantly improve the aesthetics of the bridge location. Where practical, avoid placing horizontal curves and superelevation transitions on the bridge.

e. Foundation Conditions. Investigate the soil conditions at each bridge abutment and at each pier location. Poor foundation conditions may limit possible bridge sites.

f. Existing Structures. Existing structures may limit the location of a new bridge. Provide sufficient separation between structures.

g. Environmental Considerations. Avoid or minimize the impact on environmentally or historically sensitive areas, wherever practical, in conjunction with the above guidelines.

6. Interchanges. When developing the alignment and profile of freeways near proposed interchanges, see Chapter 10 “Interchanges” for detailed guidelines.

7. Climbing Lanes. Section 6.4 discusses the warrants and design criteria for climbing lanes. For most freeways, climbing lanes are not warranted unless a drop in the level of service is significant.

17.2.4 Sight Distance

Sight distances for freeways should desirably be provided based upon the decision sight distance in areas where driver confusion may occur (e.g., within interchanges, changes in cross sections, lane drops). See Chapter 4 “Sight Distance” for additional information on stopping and decision sight distances.

17.2.5 Cross Sections

17.2.5.1 Lane and Shoulder Widths

Lane and shoulder width criteria are provided in the geometric design tables in Section 17.3. The following mitigation strategies may be considered in addition to the design exception where reduced shoulder widths are provided:

- adding advisory and regulatory signing,
- providing additional raised pavement markings,
- constructing frequent emergency pull-outs,
- using changeable overhead message signs,
- providing continuous lighting,
- incorporating truck-lane restrictions, and/or
- setting up dedicated service patrols and other incident management measures.

For more information on cross section design elements, see Section 7.2.

17.2.5.2 Typical Sections

Figures 17.2-A through 17.2-C illustrate typical cross sections for various freeway designs. Figures 17.2-A and 17.2-B provide typical sections for a rural/urban divided freeway with a depressed median. Figure 17.2-C provides a typical section for a freeway with a concrete median barrier (CMB).

17.2.5.3 Cross Slopes

Use a cross slope of 2.00 percent for up to two lanes in the same direction. Lanes beyond the second lane on one side of the crown should have a cross slope of 2.50 percent. If a roadway profile grade is less than 2.00 percent, the designer may consider using a cross slope of 2.50 percent for the outside lane to improve drainage. See Section 7.2.3.3.

For paved shoulders greater than 4 feet, provide a shoulder cross slope of 4.00 percent. For paved shoulders less than or equal to 4 feet, the cross slope should match the adjacent travel lane slope. For earth shoulders, provide a shoulder cross slope of 8.00 percent.

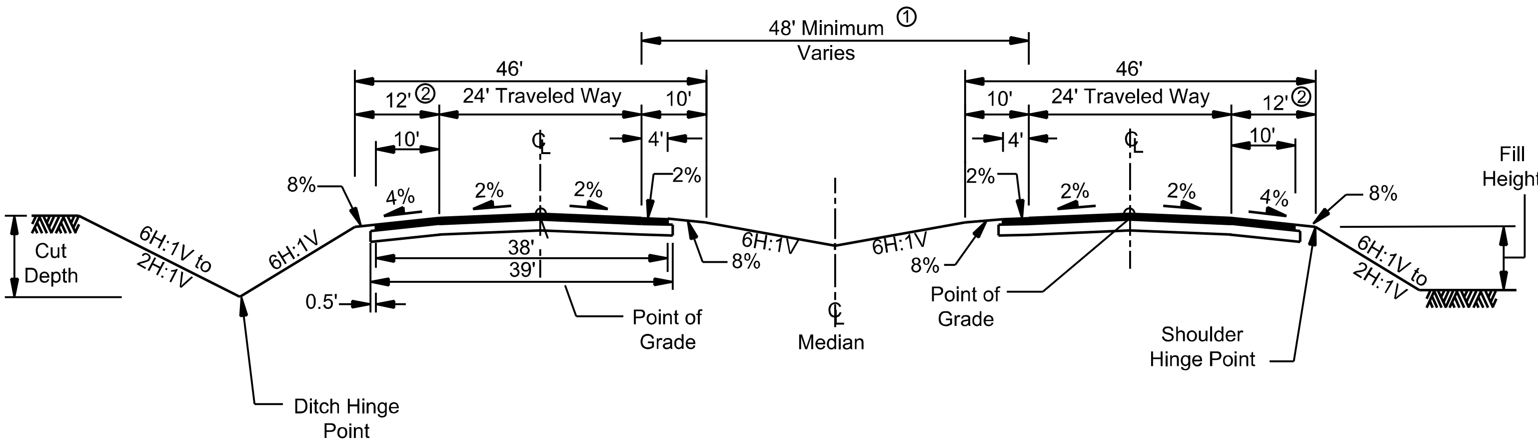
For cross slopes on bridges, see the *SCDOT Bridge Design Manual.*

17.2.5.4 Right of Way

Providing right-of-way widths that accommodate construction, drainage and proper maintenance of a collector is an important part of the overall design. Wider right of way allows for gentler side slopes, which results in reduced crash severity potential and easier maintenance operations. Right of way is typically configured to accommodate all proposed cross section elements throughout the project (e.g., travel lanes, shoulders, medians, ditches, outer slopes). If a long-range plan identifies a future widening, give consideration to accommodate a future proposed cross section. A uniform right-of-way width is preferred; however, do not base the width on the critical point of the project. A critical point may occur where the side slopes extend beyond the normal right of way, for clear areas at the bottom of traversable slopes, for wider clear areas on the outside of curves, where greater sight distance is desirable, for environmental considerations and for maintenance access.

17.2.6 Roadside Safety

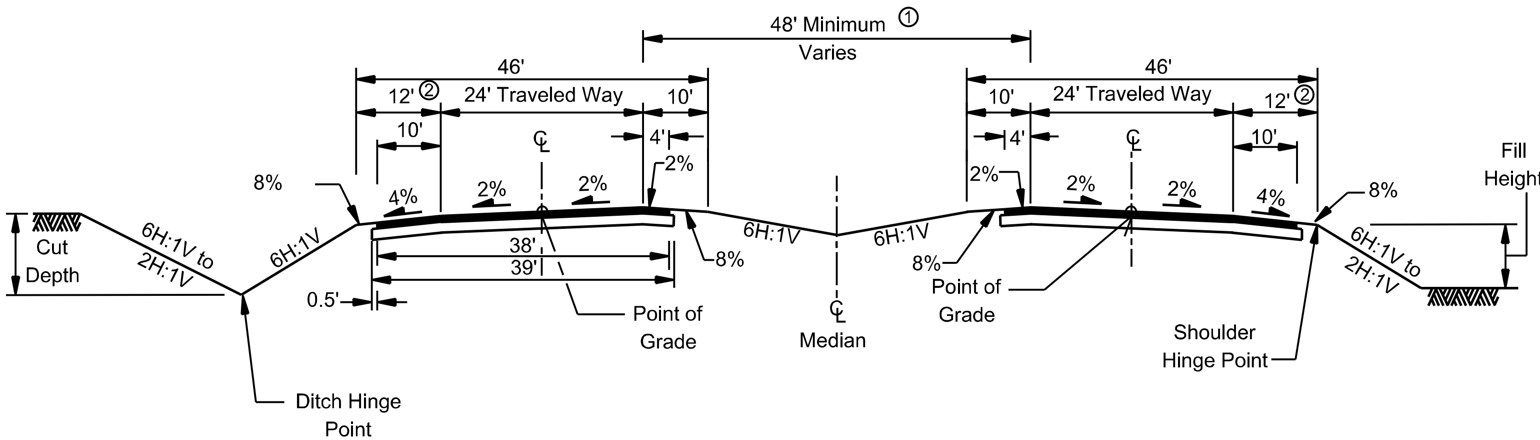
The designer should provide adequate horizontal clearance between the traveled way and roadside obstructions on freeways. The designer should provide roadside clear zones as discussed in the AASHTO *Roadside Design Guide*.



① In rural areas, a wider median may be desirable to accommodate future widening.
② Add 3.5 feet where guardrail is used.

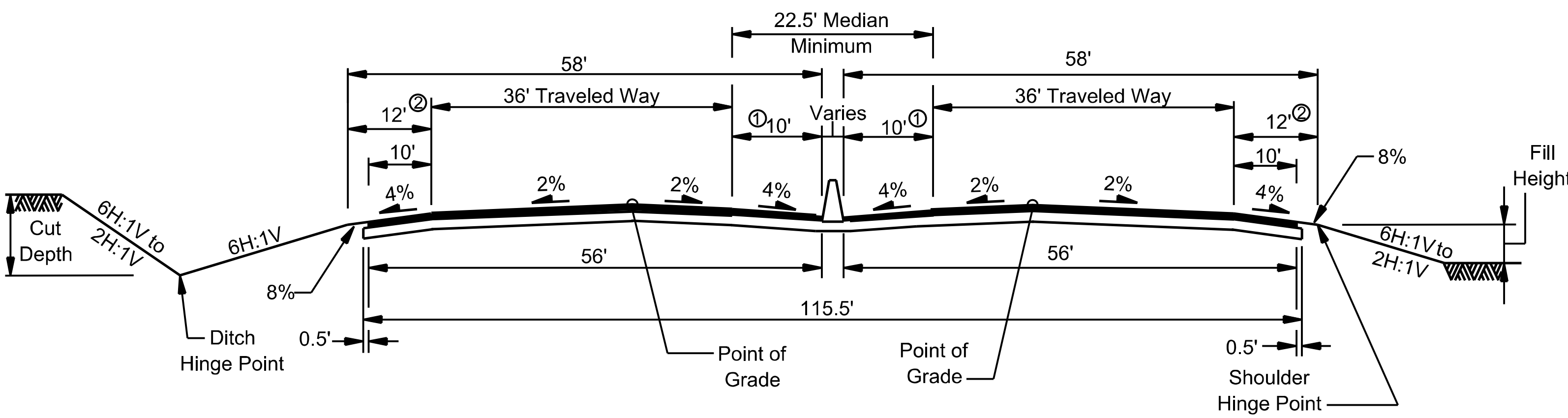
TYPICAL RURAL/URBAN FOUR-LANE DIVIDED FREEWAY
(Crowned Roadways)
Figure 17.2-A

TYPICAL RURAL/URBAN FOUR-LANE DIVIDED FREEWAY
(Uniform Cross Slope Traveled Way)
Figure 17.2-B



① Add 3.5 feet where guardrail is used.

TYPICAL RURAL/URBAN SIX-LANE DIVIDED FREEWAY
Figure 17.2-C



① When adding travel lanes to an existing median less than 48 feet, the left-shoulder width may be less than 10 feet.
② Add 3.5 feet where guardrail is used.

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17.3 TABLES OF DESIGN CRITERIA

Figures 17.3-A and 17.3-B present the Department's design criteria for rural and urban freeway projects, respectively. Figure 17.3-C presents the alignment criteria for freeway projects. The designer should consider the following when using these figures:

1. Applicability. Note that some of the cross-section elements included in the figures (e.g., flush CMB) are not automatically warranted in the project design. The values in the figures only apply after the decision has been made to include the design element in the highway cross section.

2. Manual Section References. These figures are intended to provide a concise listing of design values for easy use. However, the designer should review the *Manual* section references for more information on the design elements.

3. Footnotes. The figures include many footnotes, which are identified by a number in parentheses (e.g., **(3)**). The information in the footnotes is critical to the proper use of the design tables.

The following design tables are provided for freeways:

- Figure 17.3-A — "Geometric Design Criteria for Rural Freeways (New Construction/Reconstruction)"
- Figure 17.3-B — "Geometric Design Criteria for Urban Freeways (New Construction/Reconstruction)"
- Figure 17.3-C — "Alignment Criteria for Freeways (New Construction/Reconstruction)"

Design Element				Manual Section	Rural
Design Controls	Design Forecast Year			17.2.1	20 Years
	Minimum Design Speed **(1)**			17.2.2	70 mph
	Access Control			3.8	Full Control
	Level of Service	Desirable		3.6.4	B
		Minimum			C
Cross Section Elements	Travel Lane Width			7.2.3	12 ft
	Shoulder Width	Right	Total Width	7.2.4	12 ft
			Paved **(2)**		10 ft
		Left	Total Width		10 ft
			Paved **(3)**		4 ft
	Auxiliary Lanes	Lane Width		7.2.6	12 ft
		Shoulder Width	Total Width		12 ft
			Paved		10 ft
	Cross Slope	Travel Lane **(4)**		7.2.3	2.00%
		Auxiliary Lane **(5)**		7.2.6	2.00%
		Shoulder	Paved	7.2.4	4.00%
			Unpaved		8.00%
	Median Width	Depressed		7.4.2	Minimum: 48 ft
		Flush (CMB)			Minimum: 22.5 ft
Roadway Slopes	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V
			Ditch Type		V-Ditch
			Back Slope		6H:1V to 2H:1V
			Rock Cut **(6)**		0.25H:1V
		Fill Section	0 ft – 5 ft	7.3.2	6H:1V
			5 ft – 10 ft		4H:1V
			> 10 ft		2H:1V
	Median Slopes			7.4.2	6H:1V
	Clear Zone			-	**(7)**

GEOMETRIC DESIGN CRITERIA FOR RURAL FREEWAYS
(New Construction/Reconstruction)
Figure 17.3-A

(Continued on next page)

Design Element			Manual Section	Rural
Structures	New Bridges	Structural Capacity	7.5.1	HL-93
		Clear Roadway Width		**(8)**
	Existing Bridges to Remain in Place	Structural Capacity	7.5.1	**(9)**
		Clear Roadway Width		**(8)**
	Vertical Clearance (Freeway Under) **(10a)**	New/Replaced Overpassing Bridges **(10b)**	6.6	17 ft – 0 in
		Existing Overpassing Bridges		16 ft - 0 in
		Pedestrian Bridges		18 ft – 0 in
		Overhead Signs		17 ft – 6 in
		Overhead Utilities		Contact Utility Office
	Clearance (Freeway Over)	Railroads	6.6	23 ft – 0 in
		Underpass Width	7.5.2	Traveled Way plus Clear Zone
	Vertical Clearance (Over Water)	Navigable Water	6.6	Contact Environmental Services Office
		Major Lakes & Reservoirs (with boat traffic)		8 ft – 0 in above the high water elevation
		Rivers		2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.
		Tidal Waters		2 ft above the 10-year high water elevation including wave height.

GEOMETRIC DESIGN CRITERIA FOR RURAL FREEWAYS
(New Construction/Reconstruction)
Figure 17.3-A
(Continued on next page)

Footnotes for Figure 17.3-A

(1) Minimum Design Speed. In mountainous terrain, a minimum design speed of 55 miles per hour may be considered.

(2) Shoulder Width (Right). Where the directional distribution of trucks exceeds 250 DDHV, consider providing a 12-foot paved shoulder.

(3) Shoulder Width (Left). Where there are three or more lanes in one direction, provide a 10-foot left paved shoulder. If the directional distribution of trucks exceeds 250 DDHV, consider providing a 12-foot paved left shoulder.

(4) Travel Lane Cross Slope. On a six-lane highway crowned at the center line with CMB, use 2.00 percent for first two travel lanes adjacent to inside shoulder and 2.50 percent for third lane breaking away from outside edge of second travel lane.

(5) Auxiliary Lane Cross Slope. For auxiliary lanes adjacent to two travel lanes sloped in the same direction, use a cross slope of 2.50 percent.

(6) Side Slopes (Cut Section). Cut rock slope may vary based on a detailed geotechnical investigation.

(7) Clear Zone. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(8) Bridge Widths. Clear roadway bridge widths are measured from face to face of parapets or rails. Bridge widths are normally defined as the sum of the approach traveled way width plus total width for both shoulders. See Section 7.5.1.1 for further guidance.

(9) Existing Bridges to Remain in Place. Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(10) Vertical Clearance (Freeway Under).

(a) The clearance must be available over the traveled way, shoulders and any future widening identified in a long-range plan.

(b) Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR RURAL FREEWAYS
(New Construction/Reconstruction)
Figure 17.3-A
(Continued)

Design Element				Manual Section	Urban
Design Controls	Design Forecast Year			17.2.1	20 Years
	Minimum Design Speed **(1)**			17.2.2	Minimum: 50 mph
	Access Control			3.8	Full Control
	Level of Service	Desirable		3.6.5	C
		Minimum			D
Cross Section Elements	Travel Lane Width			7.2.3	12 ft
	Shoulder Width	Right	Total Width	7.2.4	12 ft
			Paved **(2)**		10 ft
		Left	Total Width		10 ft
			Paved **(3)**		4 ft
	Auxiliary Lanes	Lane Width		7.2.6	12 ft
		Shoulder Width	Total Width		12 ft
			Paved		10 ft
	Cross Slope	Travel Lane **(4)**		7.2.3	2.00%
		Auxiliary Lane **(5)**		7.2.6	2.00%
		Shoulder	Paved	7.2.4	4.00%
			Unpaved		8.00%
	Median Width	Depressed **(6)**		7.4.2	48 ft
		Flush (CMB) **(7)**			Minimum: 22.5 ft
Roadway Slopes	Side Slopes	Cut Section	Foreslope	7.3.2	6H:1V
			Ditch Type		V-Ditch
			Back Slope		6H:1V to 2H:1V
			Rock Cut **(8)**		0.25H:1V
		Fill Section	0 ft – 5 ft	7.3.2	6H:1V
			5 ft – 10 ft		4H:1V
			> 10 ft		2H:1V
	Median Slopes			7.4.2	6H:1V
	Clear Zone			-	**(9)**

GEOMETRIC DESIGN CRITERIA FOR URBAN FREEWAYS
(New Construction/Reconstruction)
Figure 17.3-B
(Continued on next page)

Design Element			Manual Section	Rural
Structures	New Bridges	Structural Capacity	7.5.1	HL-93
		Clear Roadway Width		**(10)**
	Existing Bridges to Remain in Place	Structural Capacity	7.5.1	**(11)**
		Clear Roadway Width		**(10)**
	Vertical Clearance (Freeway Under) **(12a)**	New/Replaced Overpassing Bridges **(12b)**	6.6	17 ft – 0 in
		Existing Overpassing Bridges		16 ft - 0 in
		Pedestrian Bridges		18 ft – 0 in
		Overhead Signs		17 ft – 6 in
		Overhead Utilities		Contact Utility Office
	Clearance (Freeway Over)	Railroads	6.6	23 ft – 0 in
		Underpass Width	7.5.2	Traveled Way plus Clear Zone
	Vertical Clearance (Over Water)	Navigable Water	6.6	Contact Environmental Services Office
		Major Lakes & Reservoirs (with boat traffic)		8 ft – 0 in above the high water elevation
		Rivers		2 ft – 0 in above the design high water. Freeboard may be increased to a maximum of 7 ft – 0 in for large rivers.
		Tidal Waters		2 ft above the 10-year high water elevation including wave height.

GEOMETRIC DESIGN CRITERIA FOR URBAN FREEWAYS
(New Construction/Reconstruction)
Figure 17.3-B
(Continued on next page)

Footnotes for Figure 17.3-B

(1) Design Speed. The design speed selected should be consistent with the anticipated operating speed of the freeway during both peak and non-peak hours.

(2) Shoulder Width (Right). Where the directional distribution of trucks exceeds 250 DDHV, consider providing a 12-foot paved shoulder.

(3) Shoulder Width (Left). Where there are three or more lanes in one direction, provide a 10-foot left paved shoulder. If the directional distribution of trucks exceeds 250 DDHV, consider providing a 12-foot paved left shoulder.

(4) Travel Lane Cross Slope. On a six-lane highway crowned at the center line with CMB, use 2.00 percent for first two travel lanes adjacent to inside shoulder and 2.50 percent for third lane breaking away from outside edge of second travel lane.

(5) Auxiliary Lane Cross Slope. For auxiliary lanes adjacent to two travel lanes sloped in the same direction, use a cross slope of 2.50 percent.

(6) Depressed Median Widths. In urban areas, existing 36-foot medians may be allowed to remain in place.

(7) Flush Median Widths (CMB). In urban areas, existing 12-foot to 14-foot medians may be allowed to remain-in-place. Where travel lanes are added to an existing median less than 48 feet, the left shoulder may be less than 10 feet.

(8) Side Slopes (Cut Section). Cut rock slope may vary based on a detailed geotechnical investigation.

(9) Clear Zone. See the AASHTO *Roadside Design Guide* for the applicable clear zones.

(10) Bridge Widths. Clear roadway bridge widths are measured from face to face of parapets or rails. Bridge widths are normally defined as the sum of the approach traveled way width plus total width for both shoulders. See Section 7.5.1.1 for further guidance.

(11) Existing Bridges to Remain in Place. Consult with the State Bridge Maintenance Engineer to determine the allowable structural capacity of bridges to remain in place.

(12) Vertical Clearance (Freeway Under).

(a) The clearance must be available over the traveled way, shoulders and any future widening identified in a long-range plan.

(b) Table value includes allowance for future overlays.

GEOMETRIC DESIGN CRITERIA FOR URBAN FREEWAYS
(New Construction/Reconstruction)
Figure 17.3-B
(Continued)

Design Element		Manual Section	Design Speed					
			50 mph	**55 mph**	**60 mph**	**65 mph**	**70 mph**	**75 mph**
Stopping Sight Distance **(1)**		4.1	425 ft	495 ft	570 ft	645 ft	730 ft	820 ft
Decision Sight Distance **(2)**		4.3	750 ft	865 ft	990 ft	1050 ft	1105 ft	1180 ft
Minimum Radii (e_{max} = 8%)		5.2	758 ft	960 ft	1200 ft	1480 ft	1810 ft	2210 ft
Superelevation Table		5.3	8%	8%	8%	8%	8%	8%
Horizontal Sight Line Offset **(3)**		5.4	30 ft	32 ft	34 ft	35 ft	37 ft	38 ft
Vertical Curvature (K-Values) **(4)**	Crest	6.5	84	114	151	193	247	312
	Sag		96	115	136	157	181	206
Maximum Grade **(5)**	Level	6.3.1	4%	4%	3%	3%	3%	3%
	Rolling		5%	5%	4%	4%	4%	4%
	Mountainous		6%	6%	6%	5%	5%	N/A
Minimum Grade **(6)**		6.3.2	Desirable: 0.5% Minimum: 0.0%					

Footnotes

(1) Stopping Sight Distance. Table values are for passenger cars on level grade.

(2) Decision Sight Distance. Table values are for speed/path/direction change on rural road, Avoidance Maneuver C. See Section 4.3 for other maneuvers.

(3) Horizontal Sight Line Offset. Table values provide the necessary middle ordinate assuming the minimum radii and stopping sight distance.

(4) Vertical Curvature. The K-values are based on stopping sight distances.

(5) Maximum Grade. Grades 1 percent steeper may be provided in constrained urban areas or where necessary in mountainous terrain.

(6) Minimum Grade. Longitudinal gradients of 0.0 percent may be acceptable on some pavements that have cross slopes that have adequate drainage. Ensure superelevation transitions are not developed in areas with 0.0 percent grade. Special ditch grades may be necessary to ensure proper project runoff management.

ALIGNMENT CRITERIA FOR FREEWAYS
(New Construction/Reconstruction)
Figure 17.3-C

17.4 INTERCHANGES/GRADE SEPARATIONS

Where there is a need to provide for the safe and efficient movement of traffic through a series of intersecting roads, it can most effectively be accomplished by providing grade separations and/or interchanges. This allows for the greatest capacity and level of service that can be achieved by providing continuous uninterrupted travel for highway users. On fully access-controlled facilities, each intersecting highway must be terminated, rerouted or provided with a grade separation or interchange. The designer must evaluate the importance of the continuity of the crossing road, feasibility of alternative routes, traffic volumes, construction costs, maintenance costs, environmental impacts, etc., to determine the most appropriate option.

17.4.1 Interchanges

Section 10.1.1 discusses several guidelines that must be considered in determining whether an interchange should be provided. In general, interchanges are provided at all freeway-to-freeway crossings and other major highways based on the anticipated demand for regional access.

Section 10.1.2 discusses the procedures for adding or revising an interchange access point to the freeway system.

17.4.2 Grade Separations

Grade separations are provided to allow two transportation facilities to cross at different elevations (e.g., highways, railroads, pedestrian crossings, bicycle paths). Separations are defined in terms of the major highway crossing over (overpass) or under (underpass) the less major facility.

The type of bridge structure provided at overpasses and underpasses is based upon site conditions and span lengths required to obtain the necessary horizontal and vertical clearances.

17.4.2.1 Justification

For each crossroad along the freeway, which is not an interchange, the designer must make a determination whether the crossroad should be closed, rerouted or provided with a grade separation; primarily comparing the respective cost and social factors for each alternative. Although cost is a major factor, the designer should review the following additional considerations:

1. Operations. Grade separations should be of sufficient number and adequate capacity to accommodate the crossroad traffic, traffic diverted to crossroads from other roads and streets terminated by the freeway, and the traffic generated by access connections to and from the mainline.

2. Locations. The location of grade separation structures is determined by assessing the need to provide for community and commercial continuity and traffic demand.

3. Site Topography. There are some sites where existing topography creates a condition in which the only rational design approach is to provide for grade separations.

4. Local Considerations. Closing the crossroad can have a significant impact on local users and the overall local road system integrity due, primarily, to changes in travel patterns. These may include:

    a. School Bus Routes. The effect of a road closure on the bus route system can be two fold. There may be an increase in the operating cost due to longer bus routes and an increase in the travel time for school children.

    b. Emergency Personnel. The financial effect of the longer detour route on emergency vehicles is generally not a concern. However, the extra response time could adversely affect the health and safety of local citizens.

    c. Businesses/Farms. Access to businesses and farms must be evaluated to ensure that these operations can continue without severe economic hardship. For businesses, the road closure can significantly affect their deliveries and the number of customers they receive (e.g., customers may be unwilling to travel the extra distance). For farmers, the road closure may require the transportation of large, slow-moving farm equipment along busy alternative facilities.

    d. Social Factors. Parks, churches, cemeteries, public facilities, and other areas or buildings of social concern generally cannot be relocated. Limited access to these facilities may create undue hardship.

    e. Land Use Planning. Consider future land use within a suburban environment to ensure adequate access and reciprocation factors are available.

When interchanges cannot be justified by traffic demands and economics, grade separations along freeways may be provided when the following conditions are met:

- there will be a decrease in traffic and/or road-user costs,
- there is a need for route continuity,
- where the intersecting road cannot be cost effectively re-routed through the use of frontage or other local roads,
- a critical need exists to maintain local access, and
- a critical need exists at railroad crossings for safety or special crossings for pedestrians or bicycle users.

17.4.2.2 General Design Considerations

Often the proposed highway grade separation (i.e., carrying the mainline over or under the crossroad) is based on topographical features or highway classification. When designing grade separations, the designer should consider the following guidelines:

1. Over versus Under. The decision on whether the freeway should be over or under the crossroad is normally dictated by topography and cost. If the topography does not favor one profile over the other, use the following guidelines to decide which highway should cross over the other:

a. Cost Effectiveness. The designer should consider the alternative that will be more cost effective to construct. Some elements to consider are the amount of embankment and excavation, span lengths, angle of skew, gradients, sight distances, alignment, vertical clearances, constructability, traffic control, right of way, access, drainage, soil conditions and construction costs.

b. Classification. Select the alternative that provides the highest design for the mainline road. Typically, the crossroad has a lower design speed and, therefore, the minor road can be designed with steeper gradients, lesser roadway widths, steeper side slopes, etc.

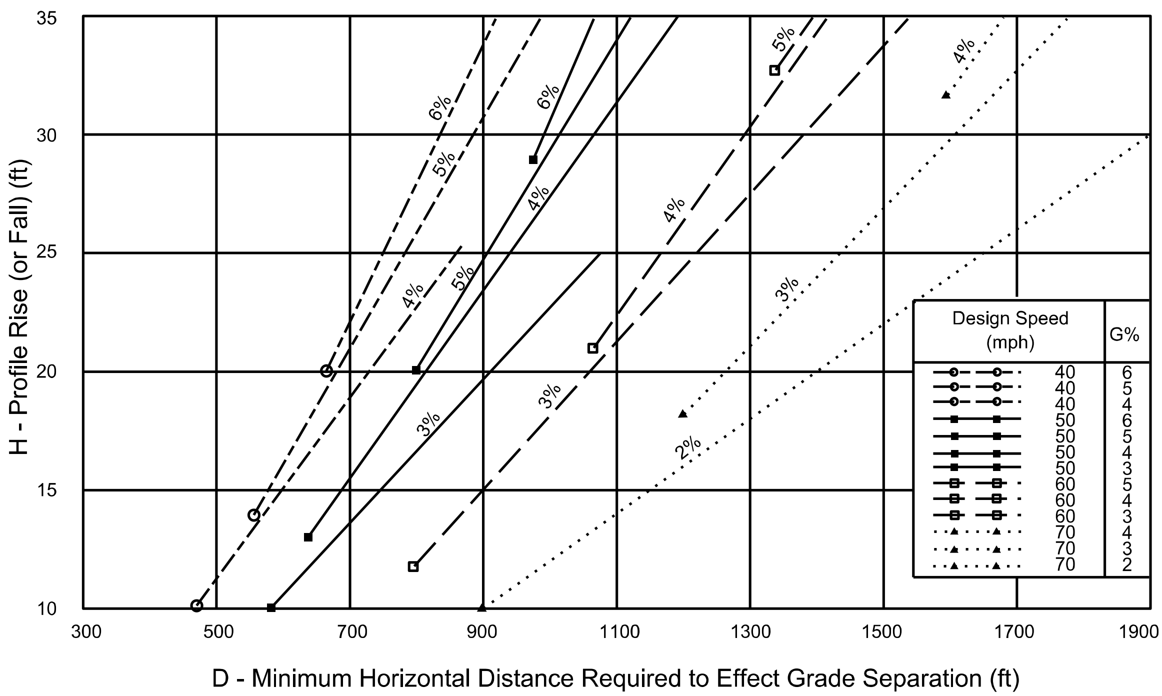
c. Future Crossings. If any crossings and/or structures are planned for a future date, the mainline should be under these future crossings. Overpasses are easier to install and will be less disruptive to the freeway when they are constructed in the future.

d. Aesthetics. Through traffic is given aesthetic preference by a layout in which the more important road is the overpass. A wide overlook can be provided from the structure and its approaches, giving drivers a minimum feeling of restriction.

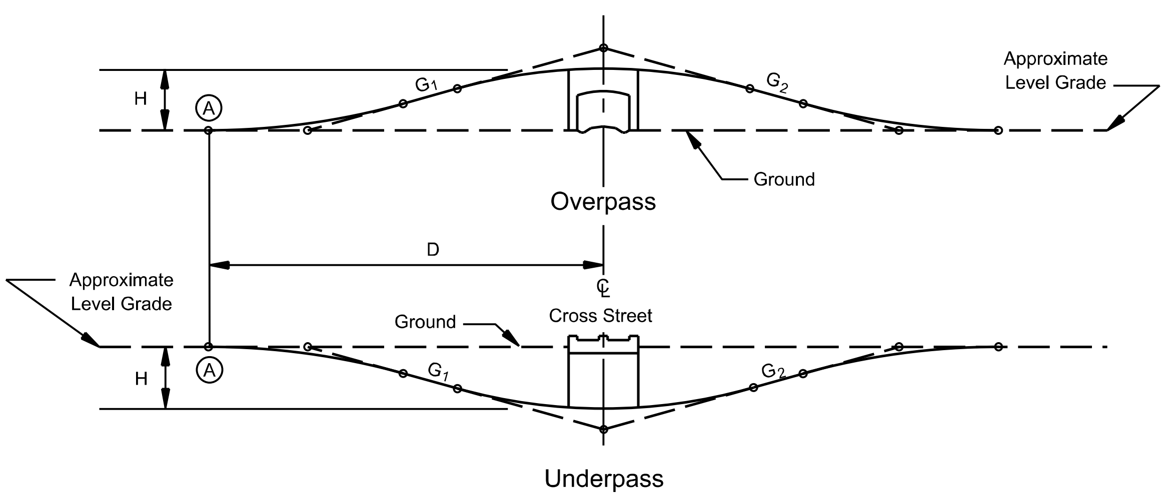
e. Turning Traffic. Where turning traffic is significant, the ramp profiles are best fitted when the major road is at the lower level. The ramp grades then assist turning vehicles to decelerate as they leave the major highway and to accelerate as they approach it. In addition, for diamond interchanges, the ramp terminal is visible to drivers as they leave the major highway.

2. Horizontal Distance. The distance required for adequate design of a grade separation depends on the design speed, the roadway gradient and the amount of rise or fall necessary to affect the separation. Figure 17.4-A can be used during preliminary design to quickly determine whether a grade separation is feasible for a given set of conditions, what gradients may be involved, and what profile adjustments may be necessary on the crossroad. Also, carefully study the sight distance requirements, because these will often dictate the required horizontal distance along the crossroad. When using Figure 17.4-A, consider the following:

a. Minimum Horizontal Distances. The plotted lines on Figure 17.4-A are derived assuming the same approach gradient on each side of the structure. However, values of D (D = minimum horizontal distance from center of structure required to effect grade separation) from Figure 17.4-A also are applicable to combinations of unequal gradients. Distance D is equal to the length of the initial vertical curve, plus one-half the central vertical curve, plus the length of tangent between the curves. Lengths of vertical curves are based on stopping sight distances. However, longer vertical curves are desirable from an aesthetic and safety standpoint. Conversely, longer curve lengths may be costlier due to increased earthwork quantities. However, these additional costs may be a less important consideration if crossroads or access points exist near the grade separation structure.



Note: Symbols on ends of lines indicate the point below which the grade is not feasible, necessitating the use of next flatter curve.



GRADE SEPARATION DETERMINATION
Figure 17.4-A

b. Maximum Gradient. The lower terminal point of each gradient line (G) on Figure 17.4-A, marked by a small symbol, indicates the distance where the tangent between the curves is zero and below which a design for the given grade is not feasible (i.e., a profile condition where the minimum central and end curves for the gradient would overlap).

c. Restricted Gradients. For the usual profile rise or fall required for a grade separation (H of 25 feet or less) (H = required profile rise or fall), do not use gradients greater than 3 percent for a design speed of 70 miles per hour, 4 percent for 60 miles per hour, 5 percent for 50 miles per hour and 6 percent for 40 miles per hour. For values of H less than 25 feet, use flatter gradients.

d. Relationship. For a given H and design speed, distance D is only shortened a negligible amount by increasing the gradient. However, the distance D varies to a greater extent for a given H and G with respect to the design speed.

e. Elevation. Considering the vertical clearance and structural depth, an elevation distance of H is typically between 23 and 25 feet for the grade separation of two highways. H is typically the same for a freeway under a railroad. For a railroad facility under a freeway, H is typically 30 to 31 feet.

f. Design Speed. To provide additional safety at rural grade separations where the crossroad passes over the freeway, consider designing the crest vertical curve with a design speed of 55 miles per hour or greater.

3. Sight Distance. In rolling topography or in rugged terrain, major-road overcrossings may be attainable only by a forced alignment and rolling gradeline. Where there is no pronounced advantage to the selection of either an underpass or an overpass, the design that provides the better sight distance (desirably passing distance if the crossroad is two lanes) on the major road should be preferred.

4. Hydrology Considerations. Carrying the major highway over without altering the crossroad grade may reduce drainage problems. In some cases, drainage issues alone may be sufficient reason for choosing to carry the major highway over rather than under the crossroad.

* * * * * * * * * *

Example 17.4-1

It is proposed that an existing crossroad be provided with an overpass over a new freeway.

Given: Crossroad Design Speed – 50 miles per hour
Difference between the proposed crossroad profile grade line and the proposed freeway profile grade line is 25.0 feet.

Problem: Determine where along the crossroad the profile grade line will need to be adjusted to provide a 25-foot profile rise.

Solution: Assume a longitudinal gradient of 4 percent. Reading into Figure 17.4-A, the minimum distance required to provide the 25-foot height distance is approximately 940 feet. Note that when using a 5 percent longitudinal gradient the distance will be approximately 900 feet.

* * * * * * * * * *

17.4.2.3 Underpass Roadway

For each underpass, the dimension, load, foundation and general site needs should determine the type of structure used for that particular location. Only the dimensional details are reviewed in this section. For guidance on the bridge design, see the *SCDOT Bridge Design Manual.*

An underpass is only one component of the total facility and should be consistent with the design criteria of the rest of the facility to the extent practical. It is desirable that the entire roadway cross section, including the median, traveled way, shoulders and roadside clear zone areas, be maintained through the structure. Possible limitations may require some reduction in the basic roadway cross section (e.g., structural design limitations, lateral clearance limitations, controls on grades and vertical clearance, limitations due to skewed crossings, appearance or aesthetic dimension relations, cost factors). However, where conditions permit a substantial length of freeway to be developed with desirable lateral dimensions, an isolated overpass along the section should not be designed as a restrictive element. In these cases, the additional structural costs are strongly encouraged to ensure consistency throughout the facility.

For a two-lane roadway or an undivided multilane roadway, the cross section width at underpasses will vary, depending on the design criteria appropriate for the particular functional classification and traffic volume. The minimum lateral clearance from the edge of the traveled way to the face of the protective barrier should be the normal shoulder width. On divided highways, the clearances on the left side of each roadway are usually governed by the median width and clear zone.

17.5 MISCELLANEOUS ELEMENTS

17.5.1 Freeway Lane Drops

Lane reductions occur when there is a sufficient change in traffic volume in which the basic number of lanes can no longer be justified. Lane drops may occur as the result of:

- the introduction of auxiliary lanes at interchanges,
- in areas where there are multiple interchanges, and/or
- collector-distributor roads necessitating multiple lanes that no longer are required to handle existing or projected traffic volumes.

Freeway lane drops should normally occur on the freeway mainline away from any other turbulence (e.g., interchange exits and entrances). Figures 17.5-A and 17.5-B illustrate right side lane drops on a freeway. In addition, consider the following criteria when designing a freeway lane drop:

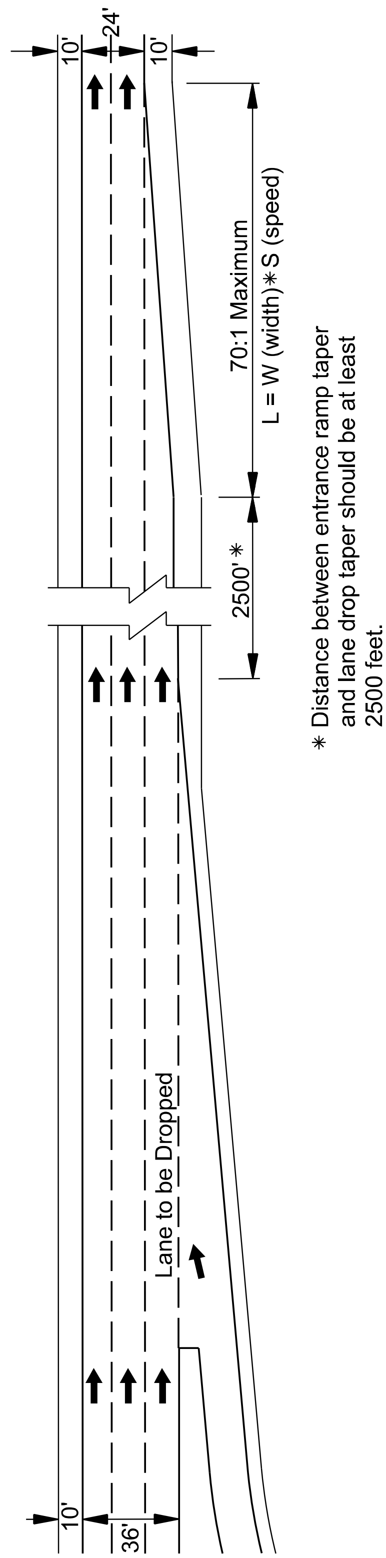
1. Location. The lane drop should occur approximately 2,500 feet beyond the previous interchange ramp; see Figure 17.5-A. The 2,500 feet allows for adequate signing and driver adjustments from the interchange, but is not so far downstream that drivers become accustomed to the number of lanes and are surprised by the lane drop. In addition, do not drop a lane on a horizontal curve or where other signing is required (e.g., an upcoming exit).

2. Transition. Transition the lane drop that involves pavement width changes over a length equal to the product of the change in lane width (W) times the design speed (S). Figure 17.5-C provides lane drop transition lengths for 12-foot lanes. The maximum transition taper rate is 70:1.

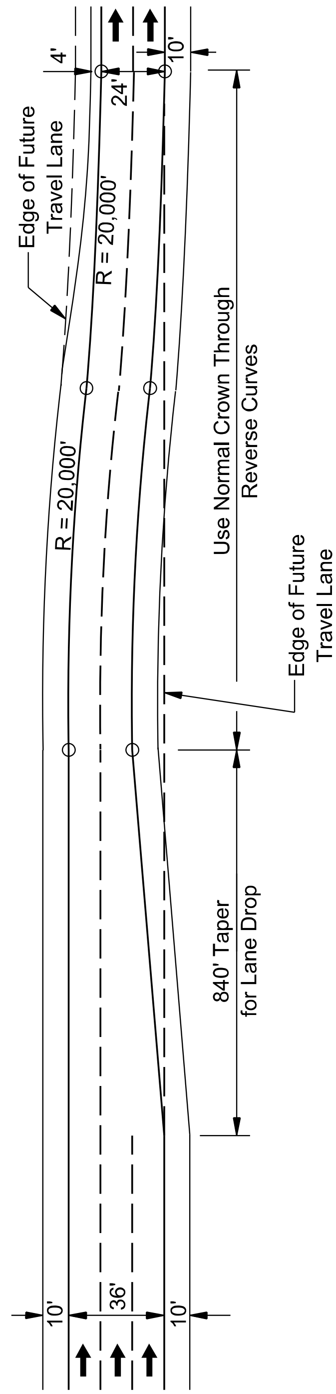
3. Sight Distance. Decision sight distance should be available to any point within the entire lane transition. See Section 4.3 for applicable decision sight distance values. This would favor, for example, placing a freeway lane drop within a sag vertical curve or at a location where the freeway lies on an upgrade, but not just beyond a crest.

4. Right-side Versus Left-side Drop. Right-side freeway lane drops are preferred due to the merging of slower vehicles and normal driver expectations. For the situation where the left lane is to be continued in the median in the future, the right-side lane drop is still preferred; see Figure 17.5-B. If a left-side lane drop is used, provide advance supplemental signing, longer taper lengths and 12-foot wide paved left shoulders beyond the area of the proposed lane drop.

5. Shoulders. Maintain the full-width right shoulder through a right-side lane drop.



TYPICAL FREEWAY LANE DROP (RIGHT SIDE)
Figure 17.5-A



TYPICAL FREEWAY LANE DROP (RIGHT SIDE)
Figure 17.5-B

Design Speed (mph)	Transition Length (feet)
55	660
60	720
65	780
70	840
75	840

TRANSITION LENGTH FOR 12-FOOT FREEWAY LANE DROP
Figure 17.5-C

17.5.2 Weaving

Design weaving segments of freeways so that the LOS within the area of weaving is consistent with the remainder of the highway. The design LOS of weaving sections depends upon their length, number of lanes, acceptable degree of congestion and relative volumes of individual traffic movements. Weaving sections may be considered as single or multiple. Detailed discussions of freeway weaving sections, relating to the operation and analysis, are contained in the *Highway Capacity Manual*.

17.5.3 Frontage Roads

17.5.3.1 General

Frontage roads are roadways adjacent to freeways and arterials that can serve many functions including providing access, maintaining traffic circulation or collecting local traffic between interchanges. The type and function of the frontage road provided is highly dependent on the function of the adjacent freeway and the area where it is located. The need for local service across and along rural freeway corridors is usually considerably less than the need along highly developed urban freeways. Frontage roads can be used on all types of freeways. Therefore, along rural freeways, frontage roads are usually intermittent and relatively short. Along urban freeways, frontage roads may extend throughout the freeway corridor to provide continuous and adjacent access to preserve the highway from subsequent development of the roadsides.

Frontage roads are outside the controlled access lines of freeways and other controlled access highways. It is preferable that frontage roads be located generally parallel to freeways on an independent right of way. For example, if the typical freeway right of way is 150 feet from the centerline of the median, then an additional 66 to 90 feet of right of way should be provided for the adjacent frontage road.

Providing adequate distance between ramp/crossroad and frontage road/crossroad intersections avoids operational and safety problems. The *SCDOT Access and Roadside Management Standards* provides the recommended distances along the crossroad between frontage roads and ramp terminals. Where right-of-way restrictions are not a consideration, the distance between the ramp terminal and frontage road should be as liberal as practical.

Where frontage roads are used on arterials without grade-separated cross roads, see Chapter 9 "Intersections" of the AASHTO *A Policy on Geometric Design of Highways and Streets* for more information.

17.5.3.2 Urban Frontage Roads

Connections between the main highway and the frontage road are an important design element in constrained urban conditions. In general, continuous frontage roads should be one way in the same direction as the adjacent freeway lanes. From an operational and safety perspective, one-way urban frontage roads are preferred to two way. Two-way frontage roads at high-volume, urban intersections may complicate crossing and turning movements. One-way operations may inconvenience local traffic to some extent, but the advantages in reducing vehicular and pedestrian conflicts at intersecting streets often fully compensates for this inconvenience. Continuous frontage roads that are parallel to the freeway permit the use of frontage roads as a backup system in case of freeway disruptions.

On facilities with lower operational speeds and one-way frontage roads, slip ramps or simple openings in a narrow outer separation may work reasonably well. Slip ramps from one-way frontage roads and freeways can be a necessary and appropriate feature in an urban corridor. Because slip ramps from a freeway to two-way frontage roads tend to induce wrong-way entry onto the freeway and may cause crashes at the intersection of the ramp and frontage road, access to the freeway must be provided only at an interchange. Do not use off or slip ramps joining two-way frontage roads because of the potential for wrong-way entry onto the freeway.

17.5.3.3 Rural Frontage Roads

Because of the lack of continuity and the type of service being provided, newly constructed frontage roads are normally two-way facilities in rural areas. Two-way frontage roads are best used where the adjoining street system is so irregular or so disconnected that one-way operation would introduce considerable added travel distance and cause undue inconvenience. Traffic operations are more complex at two-way frontage road intersections with grade separated crossroads; therefore, such intersections are generally located as far as practical from grade-separated structures and interchange ramp terminals.

Where rural freeways sever existing low-volume roads, the designer must determine if the road is to be closed, provided with a cul-de-sac, rerouted, provided with a grade separation or provided with a frontage road. This decision should be based on economics and, if necessary, through a benefit/cost study. Desirably, a freeway should be located so that a minimum number of properties are severed by its location. Realizing that this is not always practical or feasible, frontage roads are provided for access to severed properties. The designer, with the assistance of the Rights of Way Office, should conduct an economic justification study to determine if it is more economical to construct the frontage road or pay severance damages for loss of access.

17.5.3.4 Functional Classification and Design Criteria

The designer should provide the normal design elements of pavement width, cross slope, horizontal and vertical alignment, etc., consistent with the functional operation of the frontage road. That is, the same considerations relative to functional classification, design speed, traffic volumes, etc., apply to frontage roads as they apply to any other highway. The functional classification of the frontage road will be determined on a case-by-case basis.

The selection of the appropriate design criteria is based on the functional classification of the frontage road. Once the frontage road classification has been determined, the appropriate design elements (e.g., design speed, lane and shoulder widths) can be selected. For freeways, the frontage road design criteria can be found in Chapters 14 “Local Roads and Streets,” 15 “Collector Roads and Streets” and 16 “Rural and Urban Arterials.”

17.5.3.5 Outer Separations

The area between the traveled way and a frontage road or street is referred to as the outer separation. If there are no adjoining frontage roads or local streets, then these areas are referred to as borders. Basically, outer separations or borders provide areas for shoulders, slopes, drainage facilities, controlled access fencing, walls, ramps and noise abatement barriers. Outer separations may also serve as recovery areas for errant vehicles. In urban areas, the outer separation may require a reduced width due to certain restrictions (e.g., retaining walls, right-of-way restrictions). Some typical outer separations between freeways and frontage roads are shown in Figure 17.5-D.

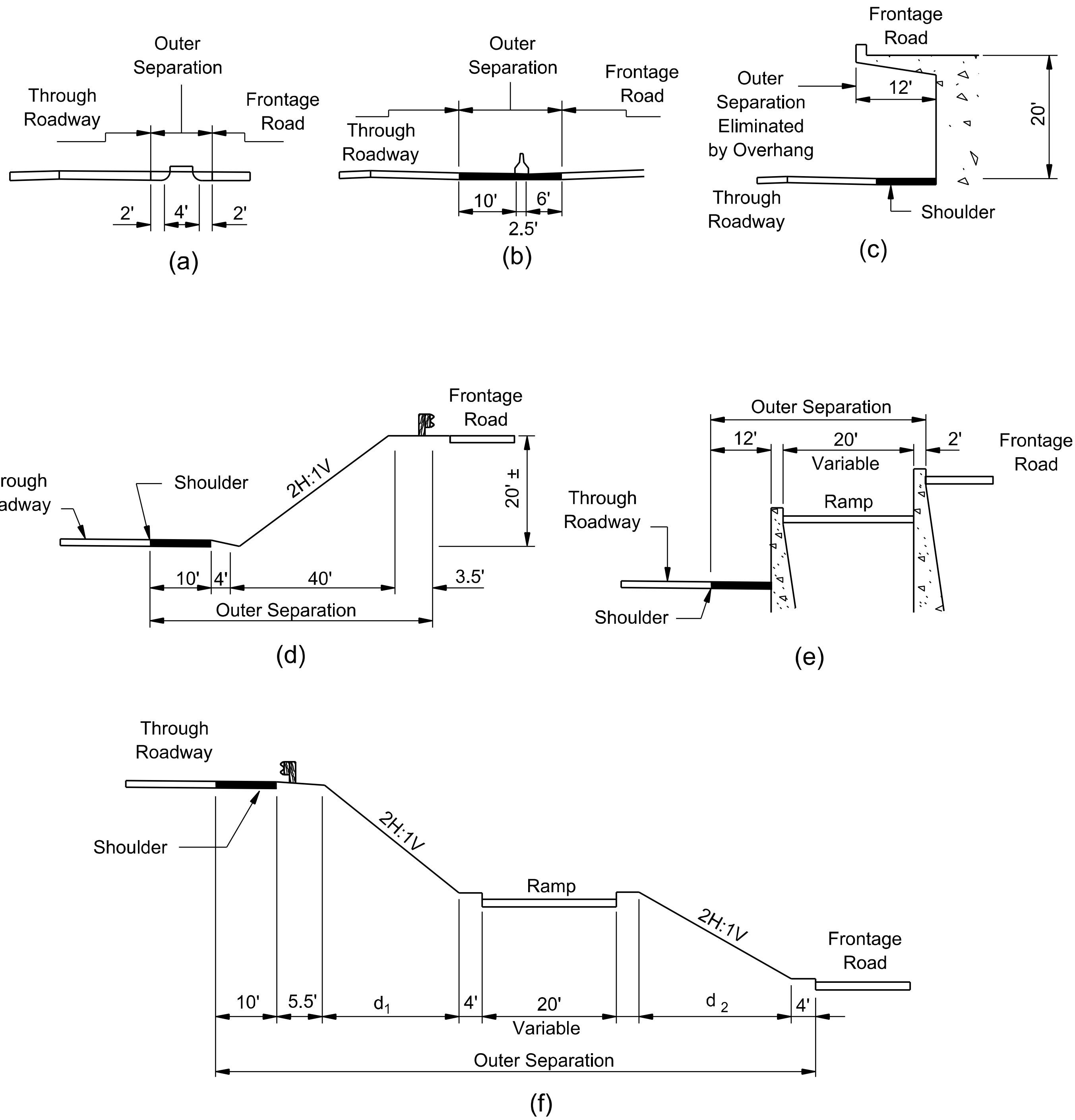
17.5.4 Pedestrians

Where planned freeway construction will divide established communities, resulting in the termination of streets and pedestrian accommodations, the designer should investigate the spacing of the remaining crossing streets and sidewalks. This should be done in conjunction with the volume of diverted pedestrian traffic and associated distances that pedestrian traffic is required to travel to determine the need for intermediate pedestrian grade separation crossings. The designer should review the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* for additional guidance.

17.5.5 Noise Barriers

The SCDOT *Traffic Noise Abatement Policy* provides SCDOT noise abatement requirements with respect to 23 CFR 772, “Procedures for Abatement of Highway Traffic Noise and Construction Noise.” SCDOT recognizes the adverse effects that highway traffic noise may have on the citizens of South Carolina and does what is practical to lessen these effects. During the project development process various noise abatement options are considered to abate noise impacts including alternative alignments or noise structures. The SCDOT Environmental Services Office is responsible for determining if noise abatement measures are required.

If a decision is made that noise barriers are required, the designer should ensure their construction will not compromise the highway safety. See Section 11.4 for guidance.



TYPICAL OUTER SEPARATIONS
Figure 17.5-D

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17.6 REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

2. *A Policy on Design Standards — Interstate System*, AASHTO, 2005.

3. *Freeway and Interchange: Geometric Design Handbook*, ITE, 2005.

4. *Highway Safety Design and Operations Guide*, AASHTO, 1997.

5. *Access and Roadside Management Standards*, SCDOT, 2008.

6. *Guide for the Planning, Design, and Operation of Pedestrian Facilities*, AASHTO, 2004.

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Chapter 18

3R PROJECTS
(Non-Freeways)

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 18
3R PROJECTS
(Non-Freeways)

A significant percentage of the Department's current and future highway program involves work on existing highways. The Department's responsibility is to realize the greatest overall benefit from the available funds. Therefore, the geometric design of projects on existing highways must be viewed from a different perspective than the design of new construction/reconstruction projects. Resurfacing, restoration and rehabilitation (3R) projects are often initiated for reasons other than geometric design deficiencies (e.g., pavement deterioration), and are often designed with right of way, financial and environmental constraints. Therefore, the design criteria for new construction are often not attainable without major and, frequently, unacceptable adverse impacts. At the same time, however, the Department must take the opportunity to make cost-effective, practical improvements to the geometric design of existing highways and streets.

For these reasons, the Department has adopted procedures and geometric design criteria for 3R non-freeway projects. They are based on a sound engineering assessment of the underlying principles behind geometric design and on how the criteria for new construction/reconstruction can legitimately be modified to apply to existing highways without sacrificing highway safety. The revised design criteria are intended to find the balance among many competing and conflicting objectives. These include the objectives of improving South Carolina's existing highways, minimizing the adverse impacts of highway construction on existing highways, and improving the greatest number of miles with the available funds.

The overall objective of the Department's criteria is to fulfill the requirements of the FHWA regulations and Technical Advisory, which govern the 3R program. These objectives are summarized as follows:

- 3R projects are intended to extend the service life of the existing facility and to return its features to a condition of structural or functional adequacy.
- 3R projects are intended to enhance highway safety.
- 3R projects are intended to incorporate cost-effective, practical improvements to the geometric design of the existing facility.

18.1 3R PROCEDURES

18.1.1 <u>Background</u>

For guidance on the background of 3R projects, the designer should review the following documents:

1. Title 23, *Code of Federal Regulations*, Part 625;

2. June 10, 1982, *Federal Register,* "Design Standards for Highways: Resurfacing, Restoration and Rehabilitation of Streets and Highways Other Than Freeways";

3. Transportation Research Board Special Report 214, *Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation*;

4. FHWA Technical Advisory T5040.28, "Developing Geometric Design Criteria and Processes for Non-Freeway RRR Projects"; and

5. NCHRP Synthesis 417, *Geometric Design Practice for Resurfacing, Restoration, and Rehabilitation.*

18.1.2 Project Types

From an overall perspective, the 3R program is intended to improve the greatest number of highway miles within the available funds for highway projects. 3R projects may include any number of the following types of improvements. This list is not all inclusive:

- providing pavement resurfacing, pavement rehabilitation and/or pavement reconstruction;
- providing lane and/or shoulder widening (without adding through lanes);
- paving shoulders;
- correcting skid hazards;
- adding a two-way, left-turn lane (TWLTL);
- adding a bike lane;
- providing intersection improvements (e.g., adding or extending turn lanes, flattening turning radii, adding channelization, realigning minor road, improving corner sight distance);
- flattening a horizontal or vertical curve;
- adding curb and gutter to an existing urban street;
- removing, widening and/or resurfacing parking lanes;
- upgrading at-grade highway/railroad crossings;
- revising the location, spacing or design of existing driveways along the mainline;
- roadway approach work associated with a bridge rehabilitation and/or widening;
- upgrading bridge rails;
- upgrading guardrail and other roadside safety appurtenances to meet current criteria;
- relocating utility poles;

- removing, providing and/or upgrading traffic control devices;
- adjusting the roadside clear zone;
- flattening side slopes;
- providing drainage improvements;
- adding or removing transit stops;
- implementing improvements to meet the Department's accessibility criteria (e.g., sidewalks and sidewalk curb ramps);
- upgrading to current access management policies; and/or
- incorporating multimodal operations.

18.1.3 Approach

The Department's approach to the geometric design of 3R projects is to adopt, where justifiable, a revised set of numerical criteria. The design criteria throughout other chapters in this *Manual* provide a frame of reference for the 3R criteria. The following summarizes the approach that has been adopted:

1. Design Speed. Section 18.2 presents guidelines for selecting 3R design speeds for arterials and collectors on the State Highway System.

2. Speed-Related Criteria. Many geometric design values are calculated directly from the design speed (e.g., vertical curves, curve radii, sight distance). The 3R design speed is used to determine these speed-related criteria. For many speed-related elements, Section 18.2 presents an acceptable threshold value for the element that is considerably below the 3R design speed. For example, if the calculated design speed of an existing crest vertical curve is within 15 mph of the 3R project design speed, the AADT is not greater than 1500 vehicles per day and there is not an adverse safety history, the existing crest vertical curve may be retained in the project design without further supporting documentation.

3. Cross Section Widths. The criteria in Chapter 15 "Collector Roads and Streets" and Chapter 16 "Rural and Urban Arterials" have been evaluated relative to the typical constraints of 3R projects. Where justifiable, the values of the cross section width criteria have been reduced. See Section 18.2 for additional discussion on cross section widths.

4. Other Design Criteria. This *Manual* contains many other details on proper geometric design techniques. These criteria are applicable to new construction and reconstruction. For 3R projects, these criteria have been evaluated and a judgment has been made on their proper application to 3R projects. Unless stated otherwise in this chapter, the criteria in other chapters of this *Manual* apply to 3R projects and should be incorporated, if practical.

18.1.4 3R Project Evaluation

The designer should evaluate available data when determining the geometric design of 3R non-freeway projects. The necessary following evaluations presented for 3R projects are based on the FHWA guidance Technical Advisory T5040.28 "Developing Geometric Design Criteria and Processes for Non-freeway RRR Projects:

1. Conduct Field Review. The designer should typically schedule a thorough field review of the proposed 3R project. One objective of the field review will be to identify potential safety concerns and potential safety improvements to the facility.

2. Document Existing Geometrics. The designer should typically review the as-built plans and combine this with the field review to determine the existing geometrics within the project limits. This review includes lane and shoulder widths, horizontal and vertical alignment, intersection geometrics and the roadside safety design.

3. Safety Analysis. The designer should conduct a safety analysis within the limits of the 3R project. Crash data is available from the Traffic Engineering Division. The designer should evaluate the following crash data analyses:

    a. Crash Rate versus Statewide Average (for that type facility). This may provide an overall indication of safety problems within the 3R project limits.

    b. Crash Analysis by Type. This may indicate if certain types of crashes are a particular problem. For example, a disproportionate number of head-on and/or sideswipe crashes may indicate inadequate roadway width. A disproportionate number of fixed-object crashes may indicate an inadequate roadside clear zone.

    c. Crash Analysis by Location. Crashes may cluster about certain locations (e.g., horizontal curve or intersection). In particular, the analysis should check to see if any sites on the Department's list of high-crash locations, as identified by the Department's crash data system, fall within the proposed project limits.

    d. *Highway Safety Manual*. The AASHTO *Highway Safety Manual* provides analytical tools and techniques for quantifying the potential effects on crashes for various improvements. The *Highway Safety Manual* also identifies factors contributing to crashes and associated potential countermeasure to address these issues.

4. FHWA Analysis Tools. FHWA provides and supports a wide range of data and safety analysis tools for State and local practitioners; see FHWA's safety website. These tools are designed to assist practitioners in understanding safety problems, link crashes to their roadway environments, and select and apply appropriate countermeasures. The tools' capabilities range from simple to complex. Some tools provide general information, while others provide complex analysis of crashes under specific conditions and/or with specific roadway features.

5. Speed Studies. It may be appropriate to review existing speed studies near the project and, if necessary, conduct a speed study to assist in determining the design speed of the 3R project. In addition, it may be desirable to conduct spot speed studies at specific

locations (e.g., in advance of a horizontal or vertical curve) to assist in the determination of geometric design improvements.

6. Traffic Volumes. The designer should examine the current and design year traffic volumes within the limits of the 3R project. This may influence the decisions on the extent of geometric improvement.

7. Early Coordination for Right-of-Way Acquisition. Significant right-of-way acquisitions are typically outside the scope of 3R projects. However, if additional right of way is required for selective safety improvements, the designer should, as early as feasible, determine which improvements will be incorporated into the project design and initiate the right-of-way acquisition process.

8. Pavement Condition. 3R projects are often programmed because of a significant deterioration of the pavement structure. The extent of pavement improvement will influence the decision on whether the project should be designed using 3R or reconstruction criteria. In addition, all 3R projects will include a pavement design that meets the Department's requirements.

   Whenever the proposed pavement improvement is major, it may be practical to include geometric improvements (e.g., lane and shoulder widening) in the project design. However, the proper level of geometric improvement is often determined by many factors other than the extent of pavement improvement. These include available right of way, traffic volumes, crash history and available funds for the project. Therefore, it may be appropriate for the 3R project to include, for example, full-depth pavement reconstruction and minimal geometric improvement, if deemed proper to meet the safety and operational objectives of the 3R program.

   Coordinate with the Director of Maintenance Office to determine acceptable pavement improvements using preventive maintenance guidelines.

9. Geometric Design of Contiguous Highway Sections. The designer should examine the geometric features and operating speeds of highway sections contiguous to the 3R project. This includes investigating whether or not any highway improvements are in the planning stages. The 3R project should provide design continuity with the contiguous sections. This involves a consideration of factors such as driver expectancy, geometric design consistency and proper transitions between sections of different geometric designs.

10. Physical Constraints. The physical constraints within the limits of the 3R project will often determine what geometric improvements are practical and cost effective. These include topography, adjacent development, available right of way, utilities and environmental constraints (e.g., wetlands, historical, culturally-sensitive areas).

11. Traffic Control Devices. All signing and pavement markings on 3R projects must meet the criteria of the *Manual on Uniform Traffic Control Devices* (MUTCD). The traffic designer is responsible for selecting and locating the traffic control devices on the project. The designer will work with the traffic designer to identify possible geometric and safety deficiencies that will remain in place and, therefore, may warrant the use of a traffic control device (e.g., a warning sign).

12. Identify Potential Countermeasures. Once potential problems have been identified, the next step involves selecting the appropriate countermeasure that will improve safety. The designer should also consider other road safety solutions beyond engineering countermeasures that can help improve safety (e.g., high visibility enforcement, public outreach, education).

13. Economics. 3R projects are intended to protect the existing economic investment and to derive the maximum economic benefit from the Department's existing highway system. Therefore, economic factors (i.e., the cost of improvement versus the anticipated benefit) are a major consideration in determining which geometric design improvements are practical and reasonable. For example, the installation of signage and rumble stripes may be an acceptable alternative to flattening a horizontal curve.

14. Potential Impacts of Various Types of Improvements. 3R projects may impact the social, economic and environmental nature of the surrounding land and development. In particular, the existing right of way may severely restrict the practical extent of geometric improvements.

Once the project evaluation is completed, the Project Manager will prepare the Project Planning Report that will recommend the proposed improvements for the 3R project.

18.2 3R GEOMETRIC DESIGN CRITERIA

18.2.1 Design Exceptions

Reference: Section 3.2

The discussion in Section 3.2 on design exceptions and variances applies equally to the geometric design of 3R projects. The only difference is that the designer will be evaluating the proposed design against the criteria presented in this chapter.

18.2.2 Design Speed

Reference: Section 3.5

Design speed is a selected speed used to determine the various geometric design features of the roadway. The designer should coordinate with the District Traffic Engineer in selecting the design speed for the 3R project. The following factors should be considered when selecting the design speed:

- new construction/reconstruction design speeds presented in Chapter 14 “Local Roads and Streets,” Chapter 15 “Collector Roads and Streets” and Chapter 16 “Rural and Urban Arterials”;
- original design speed for the roadway;
- design speeds shown the FHWA *Mitigation Strategies for Design Exceptions*; and
- context speed (e.g., urbanized areas, school zones).

18.2.3 Design Year

Reference: Section 3.6

Desirably, the design year should be 10 years from the PS&E letting date. At a minimum, it may be the current year.

18.2.4 Highway Capacity

Reference: Section 3.6

The following major factors affect the capacity analysis:

1. Design Volume. The designer should evaluate the current and design year traffic volumes within the limits of the 3R project. Give special attention to traffic movements relocated or restricted by the project.

2. Level of Service. Figure 18.2-A provides the LOS criteria for roadway segments. Depending on the project type, only the specific LOS for the traffic movements impacted may need to be evaluated. In general, the highway facility should maintain or improve the LOS for the current DHV and/or AADT.

3. Operational improvements. In some cases, 3R projects may provide operational improvements (e.g., queue length or turning radii that are not measured by LOS). In these cases, the designer will work with traffic designer regarding any highway capacity criteria for the project.

Functional Classification	Level of Service
Rural Collectors	Desirable: C Minimum: D
Urban Collectors	Desirable: C
Rural Arterials	Level/Rolling: C Mountainous: D
Urban Arterials	Desirable: C

RECOMMENDED LEVEL OF SERVICE
(3R Projects)
Figure 18.2-A

18.2.5 Cross Sections

Reference: Chapter 7 "Cross Section Elements"

18.2.5.1 Roadway Widths

Reference: Chapters 15 "Collector Roads and Streets" and 16 "Rural and Urban Arterials"

Figure 18.2-B presents lane and shoulder widths for 3R projects. In general, these widths have been established considering the minimum acceptable width from an operational and safety perspective; considering what is available for a practical improvement based on right of way and environmental impacts; and considering that, in general, it is better to improve more miles to a lower level than to improve fewer miles to a higher level. All of these considerations are consistent with the overall objectives of the Department's 3R program.

The designer should evaluate the existing roadway width with the criteria in Figure 18.2-B. If the existing width does not meet the 3R criteria, the designer should consider widening the lane and/or shoulder. If the decision is made to widen the lane or shoulder width, ensure that the width at least meets the 3R criteria. This will be sufficient for the majority of 3R projects. However, if practical, it may be appropriate to widen the roadway width to meet the new construction/reconstruction lane and shoulder width criteria in Chapters 15 "Collector Roads and Streets" and 16 "Rural and Urban Arterials."

<table>
<tr><th rowspan="2">Design Year ADT</th><th rowspan="2">Design Speed (mph)</th><th colspan="2">Rural Collectors</th><th colspan="2">Urban Collectors</th><th colspan="2">Rural Arterials</th><th colspan="2">Urban Arterials</th></tr>
<tr><th>Lane Width(1)</th><th>Shoulder Width(1)</th><th>Lane Width(1)</th><th>Shoulder Width(1)</th><th>Lane Width(1)</th><th>Shoulder Width(1)</th><th>Lane Width(1)</th><th>Shoulder Width(1)</th></tr>
<tr><td rowspan="2">1 – 750</td><td>≤ 45</td><td>10 ft</td><td>2 ft</td><td rowspan="4">D: 12 ft
M: 10 ft</td><td rowspan="5">D: 8 ft
M: 2 ft or Curb and Gutter</td><td>11 ft</td><td rowspan="4">3 ft</td><td rowspan="5">D: 12 ft
M: 11 ft</td><td rowspan="5">D: 10 ft
M: 6 ft or Curb and Gutter</td></tr>
<tr><td>> 45</td><td>10 ft</td><td>2 ft</td><td>12 ft (2)(3)</td></tr>
<tr><td rowspan="2">751 – 2000</td><td>≤ 45</td><td>11 ft (2)</td><td>2 ft</td><td>11 ft</td></tr>
<tr><td>> 45</td><td>12 ft (2)</td><td>2 ft</td><td>12 ft (2)(3)</td></tr>
<tr><td>> 2000</td><td>all</td><td>12 ft (2)</td><td>3 ft</td><td>D: 12 ft
M: 11 ft</td><td>12 ft</td><td>6 ft</td></tr>
</table>

(1) Retain existing width if existing width is greater than the value shown.
(2) Lane widths may be 1-foot less if there are less than 10 percent trucks.
(3) An existing 22-foot traveled way may be retained where the alignment is satisfactory and there is no crash pattern suggesting the need for widening.
D = Desirable M = Minimum

LANE AND SHOULDER WIDTHS
(3R Projects)
Figure 18.2-B

18.2.5.2 Cross Slopes

Reference: Chapter 7 "Cross Section Elements"

On 3R projects, it generally will be acceptable to retain the existing cross slopes. If there is an adverse crash history that indicates a problem, use the cross slope criteria for new construction/ reconstruction projects.

18.2.5.3 Bridges

Reference: Section 7.5

18.2.5.3.1 Scope of Work

Several bridges may be within the limits of the 3R project. Consult with the Bridge Maintenance Office to determine the condition and load capacity of existing bridges. The scope of work for bridges may be any of the following:

1. Bridge Replacement. Depending upon the extent of the structural deficiencies, if any, it may be economical to replace the entire bridge (i.e., superstructure, substructure and foundation).

2. Bridge Reconstruction/Bridge Deck Rehabilitation. If the existing superstructure or bridge deck is structurally deficient, but the substructure/foundation is structurally sound, the superstructure and/or bridge deck may be reconstructed or replaced as part of the 3R project. If the bridge deck is structurally sound, but its width is inadequate (i.e., the bridge is functionally deficient), the bridge deck may be rehabilitated solely to widen the

bridge deck. Bridge deck widening may then require work to the superstructure and/or substructure.

3. Existing Bridge to Remain in Place. If an existing bridge is structurally sound and if it meets the Department's design loading capacity, it is unlikely to be cost effective to improve the geometrics of the bridge. These are considered existing bridges to remain in place. However, if the geometric deficiencies are severe and/or if there has been an adverse safety experience at the bridge, it may be warranted to widen the bridge or to make other improvements.

   In some cases, only the bridge substructure (e.g., abutments, piers) and/or foundation (e.g., footings, piles) may require rehabilitative work. For applying the 3R geometric design criteria, these may be considered existing bridges to remain in place.

4. Bridge Rail Transitions. The roadway designer will evaluate the adequacy of the existing approaching guardrail transition for any needed upgrading.

18.2.5.3.2 Bridge Width

The following will apply to the evaluation and improvement to the width of bridges within the limits of a 3R project:

1. Bridge Replacement. For this scope of work, provide the full approach roadway width using new construction criteria across the bridge.

2. Bridge Reconstruction/Bridge Deck Rehabilitation. For these scopes of work, provide the full approach 3R roadway width across the bridge.

3. Existing Bridge to Remain in Place. Evaluate the existing width of bridges proposed to remain in place using the criteria from Section 7.5.1. If the existing width does not meet these criteria, the designer must either widen the bridge as part of the 3R project or pursue supporting documentation.

Evaluate all bridges that are narrower than the approach roadway width (including shoulders) for special narrow bridge treatments. At a minimum, the signing and pavement markings must meet the criteria of the MUTCD. In addition, the FHWA publication, *Mitigation Strategies for Design Exceptions* provides several mitigation strategies specifically for narrow bridges. The designer, in coordination with the traffic designer, should evaluate the value of these additional treatments at the bridge site.

18.2.5.3.3 Horizontal/Vertical Alignment

Except for bridge replacements, it is unlikely to be cost effective to improve the existing horizontal or vertical alignment for a bridge within the limits of a 3R project.

18.2.5.4 Fill or Cut Slopes

The following will apply to fill or cut slopes:

1. No Roadway Widening. Existing fill or cut slopes of 2H:1V or flatter may be retained.

2. Roadway Widening. If the lanes or shoulders are widened, this will produce a steeper fill slope or ditch foreslope, assuming the toe of fill slope or toe of backslope remains in the same location. The roadside design should be modified to provide a configuration that is the same as or flatter than the roadside cross section before the 3R project limits. At a minimum, the following will apply:

    a. Embankment Slope. Desirably, use a 6H:1V within the clear zone where a 6H:1V or flatter slope currently exists, or where the length of the improvement is greater than 0.5 mile. If a steeper slope is required, consider using a 4H:1V slope before implementing a 2H:V1 slope. Locations or situations that may warrant a 2H:1V slope are as follows:

        - roadway widening that encroaches into a wetland;
        - an area with restrictive or very costly right of way; or
        - a slope at the end of a large culvert, bridge spill slope or other location where it is desirable to protect the slope with riprap.

        The designer should analyze each location individually and use engineering judgment in selecting the slope rate.

    b. Ditch. If right of way is available, consider moving the existing ditch line and flatting slopes as much as practical. A drainage ditch in the 3R clear zone should be regraded as practical to make it traversable for an errant vehicle.

18.2.6 Right of Way

The acquisition of significant amounts of right of way is usually outside the scope of a 3R project. Where practical, secure additional right of way to allow cost-effective geometric and roadside safety improvements.

18.2.7 Horizontal Alignment

Reference: Chapter 5 “Horizontal Alignment”

The designer should determine the design speed of each existing horizontal curve within the 3R project limits. To determine the existing horizontal curve design speed, the designer should determine the applicable maximum superelevation rate for the project location. For a rural highway or an urban facility where $V \geq 50$ miles per hour, use an e_{max} of 8 percent (see Figure 5.3-B). For an urban facility where $V \leq 45$ miles per hour, use an e_{max} up to 6 percent (see Figure 5.3-C). An existing horizontal curve may be retained if the following conditions exist:

- a safety analysis does not indicate a problem at the curve site;
- the calculated curve design speed is not more than 15 miles per hour below the 3R design speed; and
- the AADT is not greater than 750 vehicles per day.

The existing radius will be retained on a curve where the above conditions are satisfied (i.e., the curve need not be evaluated further). However, proper signs and pavement markings may be necessary. Once the decision has been made to improve the curve, the designer should use the criteria in Chapter 5 "Horizontal Alignment" to determine the proper combination of radii and superelevation using the 3R design speed.

18.2.8 <u>Superelevation</u>

Reference: Chapter 5 "Horizontal Alignment"

Desirably, the curve superelevation should meet criteria for new construction; see Chapter 5 "Horizontal Alignment." On 3R projects, constraints of excessive costs often preclude the use of desirable superelevation rates. If the curve is to remain and minimum superelevation rates cannot be achieved, provide proper signing and pavement markings for the appropriate speed in accordance with the MUTCD. In some cases, reconstruction of substandard horizontal curves to larger radii may be feasible in lieu of increasing the superelevation.

18.2.9 <u>Vertical Alignment</u>

Reference: Chapter 6 "Vertical Alignment"

18.2.9.1 Grades

Reference: Section 6.3

Unless a safety analysis indicates otherwise, the maximum grade on a 3R project may be up to 2 percent steeper in level terrain or 1 percent steeper in rolling terrain than the criteria for new construction and reconstruction projects. In mountainous terrain, an existing grade may be retained.

18.2.9.2 Crest Vertical Curves

Reference: Section 6.5.1

Section 6.5.1 presents the Department's criteria for the design of crest vertical curves for new construction and reconstruction projects. The designer should use this information to determine the calculated design speed of an existing crest vertical curve and compare the calculated speed to the selected 3R design speed. The following summarizes the 3R design criteria for crest vertical curves:

1. <u>AADT < 1500</u>. In the absence of an adverse crash history, all existing crest vertical curves are acceptable without further evaluation regardless of the design speed of the vertical curve.

2. <u>AADT > 1500</u>. In the absence of an adverse crash history, all existing crest vertical curves with a calculated design speed within 15 miles per hour of the 3R design speed are acceptable. The designer should evaluate the reconstruction of the crest vertical curve if its calculated design speed is more than 15 miles per hour less than the 3R

design speed and if the crest hides from view major hazards (e.g., intersections, sharp horizontal curves, narrow bridges).

3. Angle Points. It is acceptable to retain an existing angle point (i.e., no vertical curve) where the algebraic difference between the two grades is 0.5 percent or less.

If the existing crest vertical curve satisfies these criteria, the designer typically will not need to check other details of the vertical curve (e.g., minimum length of vertical curve).

If the decision is made to flatten the crest vertical curve, the designer will desirably design the reconstructed curve to meet the criteria for new construction/reconstruction in Section 6.5.1. However, at a minimum, it is acceptable to design the crest vertical curve using a speed that is 15 miles per hour less than the 3R design speed.

In addition, consider flattening crest curves if the stopping sight distance is met, but the intersection sight distance is not available.

18.2.9.3 Sag Vertical Curves

Reference: Section 6.5.2

Section 6.5.2 presents the Department's criteria for the design of sag vertical curves for new construction and reconstruction. These criteria are based on designing the sag to allow the vehicle's headlights to illuminate the pavement for a distance equal to the stopping sight distance for the design speed. For 3R projects, the following will apply:

1. Evaluation. The comfort criteria represent the minimum criteria for the retention of an existing sag vertical curve if lighting is included. Section 6.5.2.3 presents the comfort criteria. If an existing sag does not meet the comfort criteria, then the designer should consider flattening the sag vertical curve.

2. Corrective Action. If the decision is made to flatten the sag, the designer should desirably meet the criteria for headlight sight distance in Section 6.5.2.2. At a minimum, it is acceptable to design the sag to meet the comfort criteria if lighting is included.

3. Angle Points. It is acceptable to retain an existing angle point (i.e., no vertical curve) where the algebraic difference between the two grades is 0.5 percent or less.

If the existing sag vertical curve satisfies the above criteria, the designer typically will not need to check other details of the vertical curve (e.g., minimum length of vertical curve).

18.2.9.4 Vertical Clearance

Reference: Section 6.6

Existing vertical clearance may be retained if the structure is not being reconstructed. If the bridge is being reconstructed, use the vertical clearance criteria presented in Section 6.6.

18.2.10 Intersections At-Grade

Reference: Chapter 9 “Intersections”

18.2.10.1 Intersection Sight Distance

Reference: Section 4.4

Section 4.4 presents the intersection sight distance (ISD) criteria for new construction/ reconstruction projects, which applies to 3R projects. However, for 3R projects on low-speed urban streets ($V \leq 45$ miles per hour), the location of the eye may be assumed to be 10 feet from the edge of traveled way.

18.2.11 Special Design Elements

Reference: Chapter 11 “Special Design Elements”

Chapter 11 “Special Design Elements” provides the Department’s criteria and design details for many special design elements (e.g., bus stops and turnouts, bicycle accommodations). The designer should review Chapter 11 to determine if these criteria apply to the 3R project. See the SCDOT “Americans with Disabilities Act Transition Plan” for items related to accessibility.

18.2.12 Roadside Safety

Reference: AASHTO *Roadside Design Guide*

The roadside safety criteria in the reference documents has been developed explicitly for new construction and reconstruction. This includes criteria for clear zones and barrier layout details (e.g., length of need). These criteria will apply, as practical, to the roadside safety design on 3R projects.

Achieving a roadside clear zone on a 3R project may be impractical. The roadside environment along existing highways is typically cluttered with any number of natural and man-made obstacles. To remove or relocate these obstacles can present formidable problems and public opposition, and it can be very costly. On the other hand, the designer cannot ignore the consequences for a run-off-the-road vehicle. Therefore, the designer should exercise considerable judgment when determining the appropriate clear zone on the 3R project. The most desirable objective for 3R projects will be to provide a clear zone equal to the criteria for new construction and reconstruction projects.

18.3 REFERENCES

1. *Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation*, Special Report 214, TRB, 1987.

2. *Developing Geometric Design Criteria and Processes for Non-Freeway RRR Projects*, Technical Advisory T5040.28, FHWA, 1988.

3. *NCHRP Synthesis 417, Geometric Design Practices for Resurfacing, Restoration, and Rehabilitation*, TRB, 2011.

4. *Mitigation Strategies for Design Exceptions*, FHWA, 2007.

5. *A Policy on Geometric Design of Highways and Streets*, AASHTO, 2011.

6. *Roadside Design Guide*, AASHTO, 2011.

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Chapter 19

RESERVED

SOUTH CAROLINA ROADWAY DESIGN MANUAL

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Chapter 20

QUANTITIES

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Chapter 20
QUANTITIES

In addition to preparing clear and concise construction plans as described in Chapter 21 “Procedures for Highway Plans Preparation” and Chapter 22 “Plan Sheets Preparation,” the designer needs to compile a summary of estimate quantities for the project. This information leads directly to the Engineer's Estimate, which combines the computed quantities of work and the estimated unit bid prices. An accurate estimate of quantities is critical to prospective contractors interested in submitting a bid on the project. This chapter presents detailed information on estimating quantities for highway construction projects.

20.1 GENERAL

20.1.1 <u>Quantity Summaries</u>

Computed roadway quantities, whether manually generated or computer generated, are placed on the Roadway Summary of Estimated Quantities Sheet. For more details on the Roadway Summary of Estimated Quantities Sheet, see Section 22.2.4.

The reason for summary details is multi-purposed. They are easy to review and determine that quantities are complete and accurate. It is easy to revise quantities if changes are made during final reviews or during construction. It also gives assurance that items are constructed in the proper location. It provides an easy method to determine where overruns occur and to develop as-built quantities for final payment.

Every project will require computations for project quantities. File these documents in the project folder for record keeping purposes. If the computations are being prepared electronically, then the resultant electronic files should be stored with the project’s electronic files.

20.1.2 <u>Guidelines for Preparing Quantity Summaries</u>

When preparing quantity summaries, the designer should consider the following guidelines:

1. <u>Specifications</u>. Cross check all items against the *SCDOT Standard Specifications for Highway Construction* (SCDOT *Standard Specifications*) and the Supplemental Specifications to ensure that the appropriate pay items, methods of measurement and basis of payment are used.

2. <u>Computations</u>. For the preliminary summaries, prepare a separate computation sheet for each item used on the project. Include all computation sheets in the project work file.

3. <u>Quantity Splits</u>. Some projects will require quantity splits for work conducted under various financing arrangements. The need for separate project quantities for various funding categories will be determined during the Design Field Review. For projects requiring quantity splits, organize the summary tables to readily identify each division subtotal and the total of all divisions. Show a subtotal in the summary tables for each county and each funding source.

4. Engineer's Estimate. Use only the total values from the summary tables to develop the Engineer's Estimate. All items described in the plans that are to be included in the cost estimate must be shown in the summaries.

20.1.3 Rounding

The quantity of any item provided in the summaries should match exactly with the figure provided on the computation sheets. Note any required rounding of raw estimates on the computation sheets. Unless stated otherwise, do not round the calculations until the value is incorporated into the summary tables.

On Roadway Summary Sheets, round all quantities to the nearest whole number in the unit column. The following are the only exceptions to this procedure:

1. Station Grading. Compute and list to the tenth of a station.

2. Steel Beam Guardrail. Compute and list in multiples of 12.5 feet.

3. Removal of Existing Guard Rail. Compute and list to the nearest foot.

4. Erosion Control Measures. Round and list to the nearest thousandth.

5. Precast Concrete Risers. Compute in 16-inch increments and list to the hundredth of a foot.

6. Permanent Cover, Temporary Cover, Mowing, Straw, or Hay Mulch with Tackifier. Compute and list to the thousandth of an acre.

7. Scarify, Mix, etc. Compute and list to the thousandth of a square yard.

8. Rumble Strips. Compute and list to the half of a mile.

9. Clear and Grub Material Pits. Compute and list to the tenth of an acre.

10. Class Concrete. Compute and list to the tenth of a cubic yard.

11. Signs (Construction and Permanent). Compute and list to the hundredth of a square foot.

Include the letters "NEC" (necessary) in the quantity column for all pay items where the standard unit of measurement is L.S. or lump sum.

20.1.4 Coded Pay Items

Each pay item has an official title and code number that is tied to the *SCDOT Standard Specifications*. These items are listed in the *SCDOT Standard Specifications* and the pay item spreadsheet on the Department's website. The Department uses these coded item numbers for tracking and for maintaining a historic database.

For some specialty or new items, the pay item number may not be in the database. Therefore, if the designer is unable to locate a pay item the designer will be required to conduct the following:

1. Check. The designer should ensure that there is not an actual number for the item that is currently used by reviewing the Department website or contacting the Letting Preparation Unit within the Preconstruction Support Office.

2. Specifications. The designer should review the *SCDOT Standard Specifications* and Supplemental Specifications to determine if there is a method of payment for the item. If not, see Section 21.2.2.

3. New Pay Item. If an item does not exist, the designer may request a new pay item and code number through the Letting Preparation Unit within the Preconstruction Support Office. It is important for the designer to minimize this option as much as practical. It is preferred that the design be modified slightly in order to use an existing pay item.

20.1.5 Computer Computations

For most projects, quantity estimates for earthwork and other similar items are computer generated. For small projects, it may be more efficient to manually calculate all quantities, including earthwork.

The designer should review the instruction manuals for GEOPAK to determine how to properly use the software for estimating purposes. GEOPAK can generate most project quantities. Give special consideration to the design on the computer (e.g., cell names, levels, processing procedures) to facilitate the computer generated quantities. Contact the CADD Support Group for assistance with GEOPAK.

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20.2 EARTHWORK COMPUTATIONS

20.2.1 Computer Computations

Most highway mainline earthwork computations are determined using the computer. Earthwork quantities for small projects, approaches, side roads, ditches and additional grading features may need to be calculated manually. To calculate the mainline earthwork quantities, the following information is required:

- horizontal and vertical roadway alignments;
- typical sections;
- terrain data;
- shrink and swell factors;
- cut and fill slope rates; and
- identification of sections not to be included (e.g., bridge sections).

End areas are computed mathematically for cuts and fills at each section. Earthwork computations are performed and recorded electronically. File the output data in the project file for future reference. Present these amounts on the cross sections as described in Section 20.2.2.

20.2.2 Manual Computations

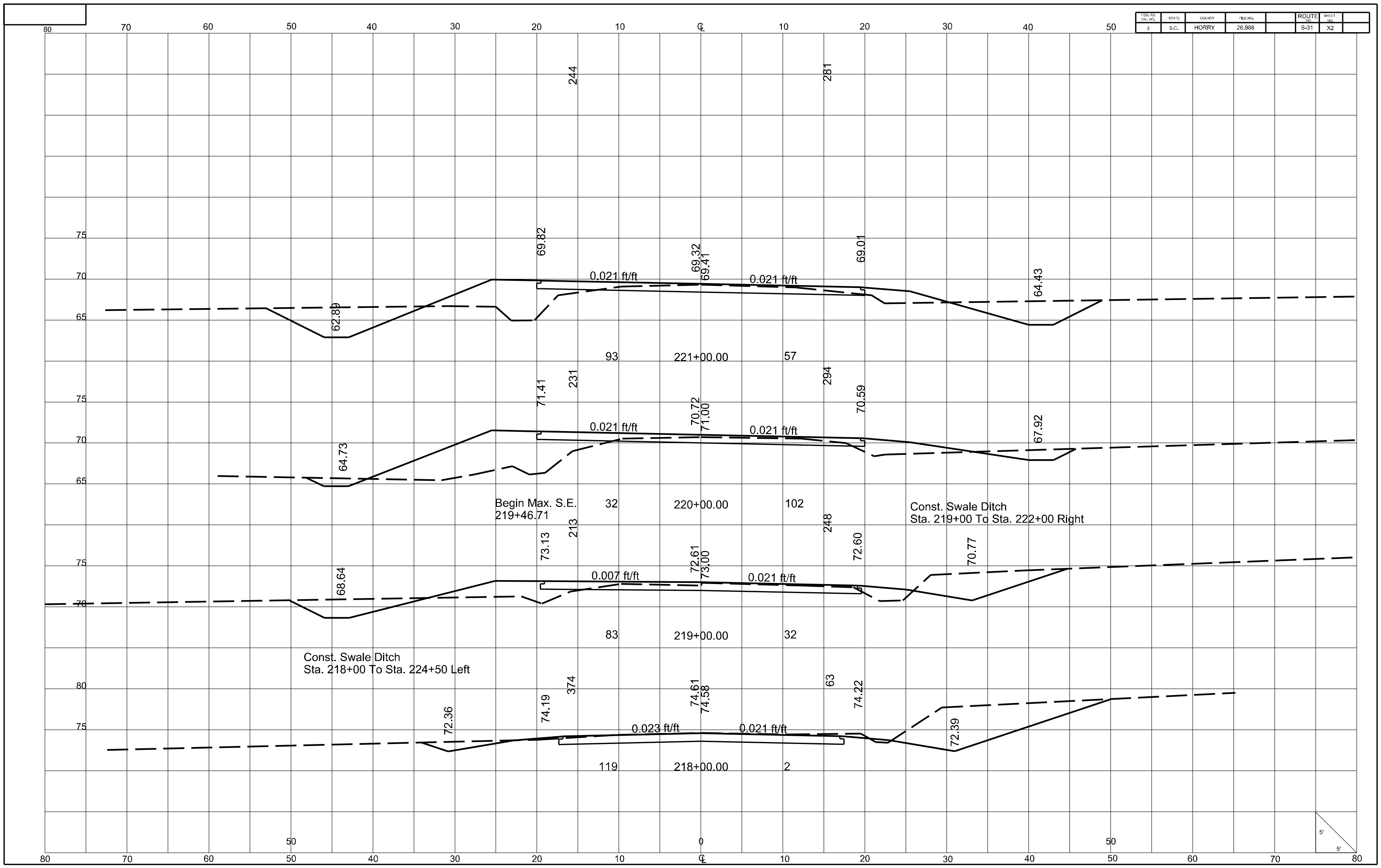
The following illustrates the procedure for calculating earthwork quantities:

1. Area Definitions. Figure 20.2-A provides a sample Cross Section Sheet used in calculating cross section end areas. The end areas that are used to compute earthwork quantities are defined by the ground lines and typical section template, and are derived planimetrically for cuts and fills at each cross section. Document the end areas on the Earthwork Computation Sheet. A sample Earthwork Computation Sheet is shown in Figure 20.2-B. Note: Show cut areas on the Cross Section Sheet by listing them horizontally to the left of the centerline and below each cross section. Show fill areas on the sheet by listing them horizontally to the right of the centerline and below each cross section. List corresponding volumes vertically left and right respectively, above each cross section, which represents the volume between cross sections.

2. Volume Computations. Determine volumes for excavation and embankment using the average-end-area formula in Equation 20.2-1. Calculate earthwork volume computations by measuring the area between the existing ground line and subgrade line (i.e., end areas), averaging the area between adjacent cross sections and multiplying by the distance between the cross sections. Volumes are noted as shown on the sample Earthwork Computation Sheet in Figure 20.2-B.

$$V = \left(\frac{A_1 + A_2}{2}\right) \times D \times \frac{1\ CY}{27\ CF} \qquad \text{(Equation 20.2-1)}$$

V = Volume, CY
$A_1 + A_2$ = Sum of cut or fill end areas of adjacent sections, SF
D = Distance between sections, FT



SAMPLE CROSS SECTIONS
Figure 20.2-A

EXCAVATION							EMBANKMENT					
					BALANCED						BALANCED	
STATION	END AREA (ft^2)	DOUBLE END AREA (ft^2)	DISTANCE (ft)	VOLUME (ft^3)	STATION	CUT (ft^3)	END AREA (ft^2)	DOUBLE END AREA (ft^2)	DISTANCE (ft)	VOLUME (ft^3)	FILL (ft^3)	F-30% (ft^3)
0+10[beg]	30				0+10		—					
1+00	209	239	90[beg]	398			—					0 / 398
+50	45	254	50	235			2	2	25	1		632
2+00	1	46	50	43			36	38	50	35		639
+50	—	1	25	1			58	94	50	87		517
3+00	14	14	25	6			57	115	50	106		385
+50	72	86	50	80			54	111	50	103		331
4+00	—	72	25	33			110	164	50	152	128	167 / 0
+50	—				4+27	796	139	249	50	231	612	796
5+00	—						110	249	50	231	103	134 / 434
+50	—						83	193	50	179		667
6+00	9	9	25	4			24	107	50	99		792
7+00	155	164	100	304		516	—	24	50	22		516 / 0
8+00	228	383	100	709	7+72	824	—				634	824
9+00	206	434	100	804		193	—					193 / 997
10+00	367	573	100	1061			—					2058
1+00	14	381	100	706			48	48	50	44		2707
2+00	6	20	100	37			277	325	100	602		1961
3+00	20	26	100	48		55	235	512	100	948	640	777 / 0
4+00	16	36	100	67	13+82	2904	183	418	100	774	2234	2904
15+00	10	26	100	48		12	78	261	100	483	134	162 / 742
+50	7	17	50	16			34	112	50	104		861
16+00	32	39	50	36			16	50	50	46		885
+50	125	157	50	145			—	16	25	7		749
17+00	131	256	50	237			1	1	25	1		514
+50	133	264	50	244			4	5	50	5		276
18+00	58	191	50	177			15	19	50	18		123 / 0
18+88[end]	48	106	88[end]	173	18+88	1088	9	24	88	39	837	1088
SHEET TOTAL				5612		5612				4317	4317	5612

EARTHWORK COMPUTATION SHEET
Figure 20.2-B

3. Computation Sheets. Figure 20.2-B is a sample Earthwork Computation Sheet used in manually calculating earthwork quantities. The columns are used for documenting stations, cross-section areas, volumes between cross sections and balance points. The Earthwork Computation Sheet facilitates the checking process by summarizing the calculation results and provides a record for the project file.

* * * * * * * * * *

Example 20.2-1

Given: Cross Sectional Area at Station 5 + 00 = 110 square feet of embankment
Cross Sectional Area at Station 5 + 50 = 83 square feet of embankment

Problem: Compute the volume of embankment.

Solution:

1. Compute the distance between the two sections.

   D = Station 5 + 50 – Station 5 + 00 = 50 FT

2. Compute the volume from Equation 20.2-1.

$$\text{Volume} = \left(\frac{110\ \text{SF} + 83\ \text{SF}}{2}\right) \times 50\ \text{FT} \times \frac{1\ \text{CY}}{27\ \text{CF}} = 178.7\ \text{CY}$$

* * * * * * * * * *

Because portions of the equation remain constant and the most frequent intervals for plotting cross sections are 100 feet and 50 feet. The same problem can be expressed as follows:

1. Sections with 100-foot spacing.

$$V = (A_1 + A_2) \times 1.851852 \qquad \text{(Equation 20.2-2)}$$

Where:

$$1.851852 = \frac{100\ \text{FT}}{2} \times \frac{1\ \text{CY}}{27\ \text{CF}}$$

2. Sections with 50-foot spacing.

$$V = (A_1 + A_2) \times 0.925926 \qquad \text{(Equation 20.2-3)}$$

Where:

$$0.925926 = \frac{50\ \text{FT}}{2} \times \frac{1\ \text{CY}}{27\ \text{CF}}$$

20.2.3 Shrink and Swell Factors

Adjust excavation and/or embankment quantities, calculated either manually or by the computer, by the appropriate shrink and/or swell factor(s). The use of more than one factor for a project may be necessary to describe the characteristics of the excavated material. The factors used in the calculations will depend on the soil type, quantity to be moved and engineering judgment. The Department's normal range of shrinkage factors is 30 percent to 40 percent. Typically, 30 percent is used west of US-1 and 40 percent is used east of US-1. If a more precise number is required, coordinate with the Geotechnical Engineer. The designer should verify the percent shrinkage or swell factor used on the project with the District Construction Engineer during the Design Field Review.

20.2.4 Determining Earthwork Balance

By using the shrinkage factor determined from past experience (e.g., various types of soil, depths of cuts and fills, type of road to be constructed), the designer should attempt to balance the earthwork. This is done in the following manner:

1. If the balance begins in a cut section, add the volume of excavation then subtract the volume of embankment plus shrinkage at each consecutive station until the two numbers are equal. (See Equation 20.2-4.) Interpolation must be used.

2. If the balance begins in a fill section, the procedure is reversed (i.e., embankment plus shrinkage minus excavation). Use interpolation to determine the correct station where the earthwork balances.

$$B = (V_1 + V_2) - (V_3 \text{ x } S) \qquad \text{(Equation 20.2-4)}$$

Where:

B = Running balance, CY
V_1 = Cut volume of the first section, CY
V_2 = Cut volume of the second section, CY
V_3 = Fill volume of the first section, CY
S = Shrinkage factor

Once the cross sectional areas have been determined and volumes computed in accordance with the instructions in Section 20.2.2, document the results on the Earthwork Computation Sheet. The Earthwork Computation Sheet depicts the method of recording the volumes of earthwork and tracking the balance between cut and fill until the values equal 0; hence, the balance point is achieved. This balance station represents a point along the alignment where the amount of excavation (i.e., cut) is equal to the amount of embankment (i.e., compacted fill) required. Several balance points may occur on one project.

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Example 20.2-2

Given: Project Length = 1878 feet
Beginning of Project = Station 0 + 10
End of Project = Station 18 + 88
Shrinkage Factor = 30 percent

Problem: Compute the balance points on the project. Begin with determining the first balance point station by developing a running balance from the beginning point. Use Figure 20.2-B as a guide in calculating the balance points for this example.

Solution:

1. Compute the volumes of the fill and cut.

   Using Equation 20.2-1, Station 1 + 00 requires 398 cubic yards of cut and 0 cubic yards of fill. Record this value in the far right column on the Earthwork Computation Sheet as shown in Figure 20.2-B.

   Using Equation 20.2-1, Station 1 + 50 requires 235 additional cubic yards of cut and 1 cubic yard of fill.

2. Compute the running balance between the cut and fill.

   B = (398 CY + 235 CY) – (1 CY x 1.30) = 632 CY

   The 1.30 factor is the shrinkage factor. Record this value in the far right column on the Earthwork Computation Sheet as shown in Figure 20.2-B.

3. Continue this procedure for the rest of the stations for excavation and embankment (see Figure 20.2-B).

   (632 CY + 43 CY) – (35 CY x 1.30) = 629 CY

4. After computing all of the volumes, the balance point can be observed to be somewhere between station 4 + 00 and 4 + 50 (i.e., the point at which a balance of 0 exists). Because the project begins with excessive excavation and is gradually being equalized by the accumulated volume of fill, a total amount of cut is documented between stations 0 + 10 and 4 + 50 equal to the value of 796 CY as shown in the Excavation Balanced Cut column. To achieve an equal amount of compacted fill, a value of 612 CY is required as shown in the Embankment Balanced Fill column (i.e., 612 CY X 1.30 = 796 CY).

   When the embankment volume values between Stations 0+10 and 4+50 are accumulated using the same procedure as in Number 2 above, it yields 484 CY. This indicates that at Station 4 + 00, the project is 128 CY short of having an exact balance.

   612 CY – 484 CY = 128 CY short

5. Adding the next embankment volume value of 231 CY to the previous accumulated embankment value of 484 CY, it indicates an excess of 103 CY beyond a balance.

484 CY + 231 CY = 715 CY embankment volume
715 CY – 612 CY = 103 CY excess

6. To compute the station at which a true balance of 0 exists, determine the proportional value between the required amount to obtain a balance (i.e., 128 CY) and the available volume of embankment material in the next section (i.e., 231 CY). Multiply the fractional value times the immediate increment for the cross sections (i.e., Station 4 + 00 to Station 4 + 50 = 50 feet).

$$\frac{128\ \text{CY}}{231\ \text{CY}} \text{ x } 50\ \text{FT} = 27.8\ \text{FT}$$

Add this distance to Station 4 + 00 to achieve the point at which a balance equals 0.

Station 4 + 00 + 27.8 FT = Station 4 + 27.8

7. The 103 cubic yards of excess is multiplied by the shrinkage factor and documented as embankment

103 CY $\times$ 1.30 = 134 CY

8. Compute the next beginning point by taking the embankment value at the next station (Station 5 + 00) and multiplying it by the shrinkage factor. Add the compacted volume carry-over found in Step 7.

(231 CY $\times$ 1.3) + 134 CY = 434 CY carry over to the next section.

9. Use this number (434 cubic yards) to continue as above to solve for the next balance point:

5,612 CY = (4,317 CY x 1.30)

Note that excavation (cut) and embankment (fill) values are documented and computed separately. The values from Figure 20.2-A for cut and fill (end areas and volumes) correspond to the values in Figure 20.2-B between stations 1 + 00 and 3 + 00. Also, note that the project is in balance. The total excavation volume of 5,612 cubic yards equals the total embankment volume of 4,317 cubic yards plus the 30 percent shrinkage value as shown in Figure 20.2-B.

The solution of the same sample problem may be performed by a computer program (the Department uses GEOPAK). Earthwork calculations are documented by summarizing the computer values at the balance points and the project termini.

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20.2.5 Unclassified Excavation

Unclassified excavation consists of roadway and drainage excavation regardless of the materials encountered or the manner in which they are removed. Unclassified excavation can be used for a variety of items that are not specifically included in the plans (e.g., cleaning outfall ditches, constructing driveways/entrances, removing asphalt pavement that is less than 2 inches thick).

See the *SCDOT Standard Specifications* for more information on when to use unclassified excavation. Unclassified excavation is calculated in cubic yards (CY).

20.2.6 Borrow Excavation

During the Design Field Review, the District Construction Engineer makes a recommendation as to whether the contractor or Department will furnish borrow pits for any unclassified excavation and base (borrow) materials that may be required. If the Department elects to provide borrow pits on a project, compute the amount of overhaul and show it on the Summary of Estimated Quantities. If the Department elects to have the borrow pits provided by the contractor, overhaul will not be a pay item on the project. See the *SCDOT Standard Specifications* for overhaul procedures.

Where an entire project has 25 cubic yards or less of borrow material, it will be itemized as Unclassified Excavation. Where there is 26 cubic yards or more of borrow material, itemize it as Borrow Excavation.

On resurfacing projects, computation of Borrow Excavation will be 200 cubic yards per mile for local roads and 400 cubic yards per mile for State roads on resurfacing projects.

Contact the Office of Materials and Research for the requirements of any cement-treated material. If needed, include the pay items of Cement Modified Subbase (8 inches uniform) and Portland Cement For Cement Modified Subbase on the Inclusion Sheet with the explanation that the quantities are due to the borrow excavation specifications. These pay items are not considered to be part of the pavement design; therefore, do not show these items on the Typical Section Sheet.

20.2.7 Site Excavation

Site excavation consists of all excavation necessary to construct the roadway to the typical sections in the plans. Site excavation is specified as a lump sum (LS) quantity and no earthwork computations are shown in the plans.

20.2.8 Muck Excavation

Contact the District or geotechnical designer for mucking quantities. Ensure the quantity of borrow to replace mucked material is included in the quantities.

20.2.9 Fine Grading

Include the pay item Fine Grading (square yards) in all plans that include Unclassified Excavation and/or Borrow Excavation where work is necessary to bring earth material into final shape. Calculate the plan quantities of Fine Grading for all subgrade areas including side roads. Only include those drives shown on the plans in the estimated quantities. The quantity of Fine Grading will be calculated in accordance with *SCDOT Standard Specifications*.

The contractor will be paid the plan quantity unless the fine grading area is changed as discussed in the *SCDOT Standard Specifications*. Only place the pay item on the Summary of Estimated Quantities Sheet. Do not use Fine Grading when the earthwork is measured and paid for as Site Excavation or Station Grading, or on resurfacing projects where the shoulder is being brought-up to grade to match the adjacent pavement.

20.2.10 Removal and Disposal of Existing Asphalt Pavement

Removal and disposal of existing asphalt pavement that is 2 inches in thickness or greater is measured and paid for by the square yard and itemized as Removal and Disposal of Existing Asphalt Pavement. All existing asphalt pavement that is less than 2 inches is paid for as Unclassified Excavation.

20.2.11 Removal and Disposal of Existing Pavement

Removal and disposal of existing pavement includes concrete pavement, concrete sidewalk, stone or concrete curbs, concrete curb and gutter and brick sidewalk. It is measured and paid for by the square yard and itemized as Removal and Disposal of Existing Pavement.

Use the item Removal and Disposal of Existing Pavement for locations where existing sidewalk or pavement must be removed in order to construct a new pedestrian ramp. Determine the number of ramps that require removal of concrete sidewalk. Multiply this value by 25 SY to determine the additional plan quantity for Removal and Disposal of Existing Pavement. The total plan quantity for removal and disposal of existing pavement should be added to any quantity determined for other work on the project.

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20.3 ROADWAY QUANTITIES

20.3.1 Base Course

20.3.1.1 Widening

Asphalt aggregate base course is typically used on projects that are widened 6 feet or less.

When widening with a different base material other than used for the overlay, calculate an extra 6 inches of width for the base course beyond the edge of the surface course.

When widening the existing pavement with the same material for the base and for the overlay (e.g., asphalt concrete surface course), do not calculate the typical extra 6 inches of width for the base course.

20.3.1.2 Sand Clay Base Course

In order to avoid conflicts during contract preparation and administration, all roads that use sand clay base course in the same project are required to essentially have the same material composition. The Program Manager must approve any necessary changes to ensure uniformity in the contract.

The designer should use Equation 20.3-1 to calculate the area of the base course needed in square yards.

$$A = (D \times W) \times \frac{1\,SY}{9\,SF} \qquad \text{(Equation 20.3-1)}$$

Where:

A = the area of the base, SY
D = the distance between sections, FT
W = the width of the base, FT

* * * * * * * * * *

Example 20.3-1

Given: Station 0 + 11 to Station 29 + 04
Width of Base = 29 feet (28 foot traveled way + 6 inches on each side)
Inclusion for Drives = 100 square yards (calculated)

Problem: Compute the estimated quantity of Sand Clay Base Course for the roadway.

Solution:

1. Compute the distance.

D = Station 29 + 04 – Station 0 + 11 = 2,893 FT

2. Compute the area using Equation 20.3-1.

$$A = (2{,}893\ \text{FT} \times 29\ \text{FT}) \times \frac{1\ \text{SY}}{9\ \text{SF}} = 9{,}322\ \text{SY}$$

3. Include the areas for the driveways.

9,322 SY + 100 SY = 9,422 SY

* * * * * * * * * *

20.3.1.3 Graded Aggregate Base Course

Earthwork quantities in the plans are for graded aggregate base course. If coquina shell base is selected, the grades will be adjusted in the field to compensate for the difference in base thickness. The quantities for Unclassified Excavation and Borrow Excavation will be adjusted prior to final payment. The designer should place a note on the General Construction Note Sheet when coquina shell base course is used.

* * * * * * * * * *

Example 20.3-2

Given: Project Termini = Station 0 + 13 to Station 55 + 52
Width of Roadway = 23 feet (22 feet + 6 inches on each side)
Inclusion for Drives = 75 square yards (calculated)

Problem: Compute quantity of Graded Aggregate Base Course.

Solution:

1. Compute the distance.

D = Station 55 + 52 – Station 0 + 13 = 5,539 FT

2. Compute the area using Equation 20.3-1.

$$A = (5{,}539\ \text{FT} \times 23\ \text{FT}) \times \frac{1\ \text{SY}}{9\ \text{SF}} = 14{,}155\ \text{SY}$$

3. Include Areas for Driveways.

14,155 SY + 75 SY = 14,230 SY

* * * * * * * * * *

When setting up quantities of base material for drives in the inclusions, only use the term Graded Aggregate Base Course in lieu of stating all three alternatives. Show the depth of base material in the inclusions.

20.3.1.4 Portland Cement for Base Course

The designer should compute and show the amount of Portland cement on the Summary of Estimated Quantities when the plans specify the use of Cement Modified Subbase, Cement Stabilized Earth Base Course and/or Cement Stabilized Aggregate Base Course. These base courses are computed and paid for in square yards and the Portland cement quantity used in these base courses is computed and paid for in tons.

Use the following procedure for computing the quantities of Portland Cement:

1. Obtain the appropriate "D" factor (density of material) from the Office of Materials and Research.

2. Obtain the appropriate percentage factor for Portland cement ratio from the Office of Materials and Research for each base course in the project.

3. Figure the estimated quantities by applying Equation 20.3-2 and the following steps:

   a. Convert plan quantities of the base courses noted above from square yards to cubic feet.

   b. Compute the product of cubic feet of material, the density of material (D) and the ratio (percentage) of cement to the item.

   c. Convert the answer found in Step b. to tons by dividing by 2000.

$$\text{Ton} = (\text{V x D x \%}) \text{ x } \frac{1\,\text{TON}}{2{,}000\,\text{LBS}} \qquad \text{(Equation 20.3-2)}$$

Where:

V	=	Volume of Base Course, CF
D	=	Density of Material, LBS/CF
%	=	Percent by Weight of Cement

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<u>Example 20.3-3</u>

<u>Given</u>: Cement Stabilized Earth Base Course – 6 inches Uniform = 3900 square yards
Density of Material = 125 pounds per cubic foot (from Office of Materials and Research)
Percentage of Cement by Weight = 6 percent

<u>Problem</u>: Compute the estimated quantity of Cement Stabilized Earth Base Course – 6 inches Uniform.

Solution:

1. Convert square yards to square feet by multiplying by 9. Multiply by the depth (6 inches uniform) of mix to get the volume in CF.

$$3{,}900 \text{ SY x } \frac{9 \text{ SF}}{1 \text{ SY}} \text{ x } 0.5 \text{ FT} = 17{,}550 \text{ CF}$$

2. Use Equation 20.3-2 to solve for estimated quantity.

$$17{,}550 \text{ CF x } \frac{125 \text{ LBS}}{1 \text{ CF}} \text{ x } 0.06 \text{ x } \frac{1 \text{ TON}}{2{,}000 \text{ LBS}} = 65.81 \text{ TON}$$

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20.3.1.5 Prime Coat

The prime coat is calculated and added to the list of estimated quantities for projects using graded aggregate base course and sand clay base course. The rate of application is 0.27 gallons per square yard. When computing the square area, include the entire width of the base course.

$$P = A \times R \qquad \text{(Equation 20.3-3)}$$

Where:

P	=	Volume of prime coat, GAL
A	=	Area of base course, SY
R	=	Rate of application for prime coat, GAL/SY

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Example 20.3-4

Given: Area for Base Course = 14,155 square yards (from Example 20.3-2)
Rate of Application = 0.27 gallons per square yard

Problem: Compute the estimated quantity of the prime coat.

Solution: Use Equation 20.3-3.

$$14{,}155 \text{ SY x } \frac{0.27 \text{ GAL}}{1 \text{ SY}} = 3{,}822 \text{ GAL} \text{ of prime coat}$$

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20.3.1.6 Hot Mix Asphalt Aggregate Base Course

The rate of application for hot mix asphalt aggregate base course is determined in the Design Field Review or in the pavement design.

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Example 20.3-5

Given: Station 0 + 13 to Station 55 + 52
Width of Roadway = 23 feet
Rate of Application = 800 pounds of hot mix asphalt per square yard

Problem: Compute the estimated quantity for Hot Mix Asphalt Aggregate Base Course Type 1.

Solution:

1. Compute the distance.

Station 55 + 52 – Station 0 + 13 = 5,539 FT

2. Compute the area using Equation 20.3-1.

$$A = (5{,}539\,\text{FT} \times 23\,\text{FT}) \times \frac{1\,\text{SY}}{9\,\text{SF}} = 14{,}155\ \text{SY}$$

3. Multiply the area times the rate of application and convert square yards to tons.

$$14{,}155\ \text{SY} \times \frac{800\ \text{LBS}}{1\,\text{SY}} \times \frac{1\,\text{TON}}{2000\ \text{LB}} = 5{,}662\ \text{TON}$$

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20.3.2 Flexible Pavements

20.3.2.1 Hot Mix Asphalt Intermediate Course

This course consists of an intermediate course composed of mineral aggregate and asphalt binder. The rate of application for the intermediate course is determine in the Design Field Review or in the pavement design.

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Example 20.3-6

Given: Station 0 + 13 to Station 55 + 52
Width of Intermediate Course = 22 feet
Rate of Application = 225 pounds Intermediate Course per square yard
Overruns = 5 percent
Build-up for Intermediate Course = 50 tons

Problem: Compute the quantity Hot Mix Asphalt Intermediate Course.

Solution:

1. Compute the distance.

Station 55 + 52 – Station 0 + 13 = 5,539 FT

2. Compute the area using Equation 20.3-1.

$$A = (5{,}539\,FT \times 22\,FT) \times \frac{1\,SY}{9\,SF} = 13{,}540\ SY$$

3. Multiply the area times the rate of application and convert square yards to tons.

$$13{,}540\ SY \times \frac{225\ LBS}{1\,SY} \times \frac{1\,TON}{2000\ LBS} = 1{,}523\ TON$$

4. Add build-up for Intermediate Course.

1,523 TON + 50 TON = 1,573 TON

5. Add overruns.

$$(1{,}573\ TON \times 0.05) + 1{,}573\ TON = 1{,}652\ TON$$

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20.3.2.2 Liquid Asphalt Binder in Paving Mixture

Liquid asphalt binder is included for all asphalt paving mixtures. Consult with the Office of Materials and Research for the recommended values for computing these quantities and for the specific binder to be used. The *Guidelines for Hot Mix Asphalt Section Table* can be found on the Office of Materials and Research webpage.

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Example 20.3-7

Given: 1273 Tons of ACSC Type C Surface Course.

Problem: Compute the estimated quantity of Liquid Asphalt Binder required.

Solution: Multiply the surface course by the percentage provided in the *Guidelines for Hot Mix Asphalt Section Table* found on the Office of Materials and Research webpage.

1273 TON x 0.06 = 76.4 TONS Liquid Asphalt Binder

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20.3.2.3 Full Depth Asphalt Pavement Patching

Full depth asphalt pavement patching is limited to a uniform depth. The designer should provide an estimated quantity of Maintenance Stone for stabilizing the patch area below the designated depth. Use a rate of 25 tons/100SY.

20.3.2.4 Asphalt Weight and Thickness

The conversion factors for the weight and thickness of hot mix asphalt base and surface courses are 110 LBS/SY (approximate) for 1-inch thick and 0.91 Equivalent inches/100 LBS/SY.

20.3.2.5 Hot Mix Asphalt Surface Course

Recommendations from the Design Field Review or the pavement design will provide the rate of application for the surface course. Consult with the Office of Materials and Research for the project pavement design. The *Guidelines for Hot Mix Asphalt Section Table* can be found on the Office of Materials and Research webpage.

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<u>Example 20.3-8</u>

<u>Given</u>: Station 0 + 13 to Station 55 + 52 = 5,539 feet
Traveled Way Width = 22 feet
Inclusion for Drives = 66 tons
Rate of Application = 175 pounds of Hot Mix Asphalt per square yard
Overruns = 2 percent

<u>Problem</u>: Compute the estimated quantity for Hot Mix Asphalt Surface Course.

<u>Solution</u>:

1. Compute the area and convert to square yards.

$$A = (5{,}539\,\text{FT} \times 22\,\text{FT}) \times \frac{1\,\text{SY}}{9\,\text{SF}} = 13{,}540\ \text{SY}$$

2. Convert square yards to tons.

$$13{,}540\ \text{SY} \times \frac{175\ \text{LBS}}{1\,\text{SY}} \times \frac{1\,\text{TON}}{2{,}000\ \text{LBS}} = 1{,}185\ \text{TON}$$

3. Include the quantity for driveways.

$$1{,}185\ \text{TON} + 66\ \text{TON} = 1{,}251\ \text{TON}$$

4. Include overruns.

$$(1{,}251 \times 0.02) + 1{,}251 = 1{,}276\ \text{TON}$$

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20.3.2.6 Bituminous Surfacing

The type of bituminous surfacing for surface course is provided during the Design Field Review or in the pavement design.

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Example 20.3-9

Given: Station 0 + 11 to Station 29 + 04 = 2893 feet
Traveled Way Width = 28 feet
Inclusions for Drives = 500 square yards

Problem: Compute the estimated quantity for the surfacing.

Solution:

1. Compute the area and convert to square yards.

$$A = 2{,}893 \text{ FT x } 28 \text{ FT x} \frac{1 \text{ SY}}{9 \text{ SF}} = 9{,}000 \text{ SY}$$

2. Include the area for the driveways.

$$9{,}000 \text{ SY} + 500 \text{ SY} = 9{,}500 \text{ SY}$$

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20.3.3 Rigid Pavements

Portland cement concrete pavements are uniform thickness as determined by the Office of Materials and Research and are paid for in square yards.

20.4 TRAFFIC ITEMS

For assistance in traffic related calculations, contact the traffic designer. Make all submittals to Traffic Engineering in accordance with Preconstruction Design Memorandum PCDM-01 “Coordinating with Traffic Engineering.”

20.4.1 Pavement Markings

Pavement marking quantities are to be provided for every road and bridge project. Coordinate with the traffic designer to determine how to calculate quantities.

20.4.2 Signing

Signing plans are prepared on all control of access construction projects. For other projects, signing plans are not required, but may be prepared by headquarters or District Office staff for implementation at the end of the project or by SCDOT forces.

20.4.3 Traffic Signals

Coordinate with the traffic designer to determine information to provide in the plans.

20.4.4 Work Zone Traffic Control

Coordinate with the traffic designer to determine quantities. The roadway designer will calculate quantities for removing temporary work items used for staging.

20.4.5 Intelligent Transportation Systems

Contact the traffic designer for assistance with ITS quantities.

20.4.6 Permanent Construction Signing

Calculate the quantities based on the *SCDOT Standard Drawings* for all projects.

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20.5 DRAINAGE

The designer should be knowledgeable of all drainage policy criteria affecting other elements of roadway design (e.g., establishing final profiles in flood plains, culvert headwater control, bridge freeboard at major stream crossings). Storm drainage design, culvert design, hydraulic data required on Roadway/Bridge Plans and other drainage criteria are addressed in the Department's *Requirements for Hydraulic Design Studies,* which is available on the SCDOT website.

20.5.1 Riprap

The Department uses multiple classes of riprap that are described in the *SCDOT Standard Specifications*. All notes in the plans pertaining to riprap should describe the quantity and class of riprap used.

Where the hydraulic designer specifies a class of riprap, the class is noted on the plans at each location. If a hydraulic study is not performed, the roadway designer may make recommendations for the class of riprap.

When calculating the quantity for riprap, assume a weight of 3400 LBS/CY for riprap.

20.5.1.1 Riprap for Ditches

The following discussion and Example 20.5-1 provide the procedures for computing riprap, in tons, for a ditch:

1. Determine the length of the ditch.

2. Calculate the perimeter of the ditch cross section.

$$P = B + 2D\sqrt{Z^2 + 1} \qquad \text{(Equation 20.5-1)}$$

Where:

P = perimeter, FT
B = base of ditch, FT
D = depth of ditch, FT
Z = rate of side slope of ditch

Note: This procedure can also be used to calculate riprap for a V-ditch by setting B equal to 0.

3. Calculate the depth of riprap by multiplying 2 times D_{50}. D_{50} is the diameter of the particle size in which 50 percent of the material is smaller. See the *SCDOT Standard Drawings* to determine D_{50}.

4. Multiply the perimeter by the depth to get the cross-sectional area.

5. Multiply the cross-sectional area by the length of the ditch to obtain the volume of the riprap.

6. Convert the volume of the riprap to tons.

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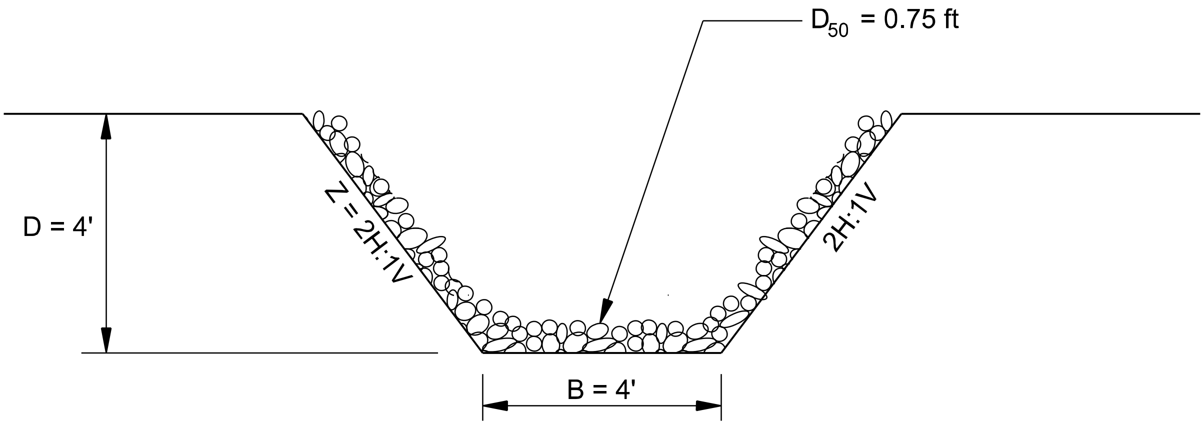
Example 20.5-1

Given: A flat-bottom ditch with side slopes of 2H:1V and a bottom width of 4 feet is constructed from Station 201 + 00 to Station 207 + 00.

D_{50} = 0.75 FT, See the *SCDOT Standard Drawings* for additional guidance.



FLAT BOTTOM DITCH
Figure 20.5-A

Problem: Compute the necessary quantity of riprap, in tons.

Solution:

1. Compute the length of the ditch.

STA. 207 + 00 – STA. 201 + 00 = 600 FT

2. Compute the perimeter of the ditch.

$$P = B + 2D\sqrt{Z^2 + 1} = 4 + 2(4)\sqrt{2^2 + 1}$$

$$= 21.9 \text{ ft}$$

3. Compute the volume of the riprap, in cubic feet.

Volume = P x L x (2 x D_{50})
Volume = 21.9 FT x 600 FT x (2 x 0.75 FT) = 19,710 CF

4. Convert cubic feet of riprap to tons.

$$19{,}710\text{ CF} \times \frac{1\text{ CY}}{27\text{ CF}} \times \frac{3400\text{ LBS}}{1\text{ CY}} \times \frac{1\text{ TON}}{2000\text{ LBS}} = 1241\text{ TON}$$

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20.5.1.2 Hand Placed Riprap

Unless otherwise specified on the plans, hand placed riprap is Class B. When specified, Class A riprap may be placed by hand.

20.5.1.3 Dumped Riprap

Dumped riprap may be Class A, B, C, D, E or F and is shown on the plans. In the event that the class for dumped riprap is not indicated on the plans, it can be assumed to be Class C.

20.5.1.4 Foundation Riprap

The class for foundation riprap is designated on the plans. In the event that the class for dumped riprap is not indicated on the plans, it can be assumed to be Class C.

20.5.1.5 Riprap for Bridge Ends

In addition to the *SCDOT Standard Drawings*, guidance on placement of riprap for bridge ends is provided on SCDOT internet site.

20.5.1.6 Box Culverts and Pipe End Treatments

Coordinate with the hydraulic designer to determine locations where riprap is required. The designer should then calculate riprap quantities based on the dimensions shown in the *SCDOT Standard Drawings*.

20.5.1.7 Geotextile Fabric for Erosion Control Under Riprap

Geotextile for erosion control is to be used under all riprap. The geotextile class will be provided by the Office of Materials and Research.

20.5.2 Drainage Structures

20.5.2.1 Pipe Lengths

The length is obtained by adding the centerline length of each run of pipe between drainage structures or to the completed end of pipe at end treatments. Calculate the pipes lengths along the pipe slope. The individual pipe length will be accurate to the nearest foot. Round the total pipe quantity up to the nearest length divisible by four. See Supplemental Technical Specifications SC-M-714 for further guidance.

20.5.2.2 Precast Concrete Structures

See the *SCDOT Standard Drawings* for dimensions of precast structures.

20.5.2.3 Extra Depth of Box

When the depth of a catch basin, drop inlet, manhole, junction box or spring box is greater than 6 feet, the quantity for the pay item Extra Depth of Box is the depth of excavation for the drainage structure in excess of 6 feet and is measured by the LF, complete and accepted.

20.5.2.4 Crossline and Sideline Pipes

Pipe quantities will be the linear measurement from end to end of the pipe through tees, wyes, elbow, bends, reducers, increasers and beveled ends. For additional information, see SCDOT Supplemental Technical Specification SC-M-714.

20.5.2.5 Pipe End Treatment

1. <u>Straight Headwall for Circular Pipe</u>. Calculate in accordance with the *SCDOT Standard Drawings*, SCDOT Supplemental Technical Specification SC-M-714 and Special Provisions

2. <u>Beveled End</u>. Calculate in accordance with the *SCDOT Standard Drawings*, SCDOT Supplemental Technical Specification SC-M-714 and special provisions

3. <u>Load Caring Grate</u>. Calculate in accordance with *SCDOT Standard Drawings*. See the AASHTO *Roadside Design Guide* for additional information.

20.5.2.6 Reinforced Concrete Box Culvert

The structural designer will provide a summary of estimated quantities to the roadway designer. For reinforced concrete box culverts, include the pay items Structure Excavation for Culvert (CY), Concrete for Structures-4000P (CY) and Reinforce Steel for Structures (Roadways) (LBS).

20.5.3 <u>Erosion and Sediment Control Best Management Practices</u>

Erosion control Best Management Practices (BMP) consist of materials, structures and construction methods that minimize the adverse impacts of the movement of water over the project site. Sediment control BMPs consist of materials, structures and construction methods that trap and minimize the adverse impacts of eroded soil particles. In typical construction applications, these erosion and sediment control BMPs are temporary measures.

Land development should be planned to control and limit erosion and sediment discharges from construction sites using BMPs. The goals of erosion and sediment control BMPs are to:

- minimize the extent and duration of disturbed soil exposure;
- protect off-site and downstream locations, drainage systems and natural waterways from the impacts of erosion and sedimentation;
- limit the exit velocities of the flow leaving the site to non-erosive or pre-development conditions; and
- design and implement an ongoing inspection and maintenance plan.

The SCDOT will not pay for BMP that are not constructed and maintained according to the *SCDOT Standard Drawings*.

20.5.3.1 Erosion Control BMP

The SCDOT Supplemental Technical Specifications, *SCDOT Standard Drawings* and *SCDOT Standard Specifications* provide the criteria for determining the quantities for erosion control items. The designer should note the following:

1. Seeding. See the latest edition of the SCDOT Supplemental Technical Specification SC-M-810.
2. Rolled Erosion Control Products. See the latest edition of the SCDOT Supplemental Technical Specification SC-M-815.
3. Diversion Dikes. See the *SCDOT Standard Drawings*.
4. Silt Ditches. See the *SCDOT Standard Drawings*.
5. Pipe Slope Drains. See the *SCDOT Standard Specifications* and the *SCDOT Standard Drawings.*
6. Anionic Polyacrylamides (PAM). See the latest edition of the SCDOT Supplemental Technical Specification *(to be prepared).*
7. Riprap. See Section 20.5.1.
8. Course Aggregate. Unit weight is 4500 LBS/CY.

20.5.3.2 Sediment Control BMP

Coordinate with the hydraulic designer to determine necessary items for sediment control. Reference the *SCDOT Standard Drawings* and Supplemental Technical Specification for guidance concerning quantity computations.

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20.6 GEOTECHNICAL

The geotechnical designer will provide geotechnical details and quantities to the roadway designer. This may include, but is not limited to, earth retaining structures, ground modifications and other geotechnical improvements. See the *SCDOT Geotechnical Design Manual* for additional guidance.

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20.7 MISCELLANEOUS ITEMS

20.7.1 Guardrail

All projects that have guardrail end treatments are identified in the plans as an End Treatment – Type T. The pay unit for guardrail end treatments is each. End terminal Type B is calculated as each.

The redirecting, non-gating portion of the guardrail end treatment can be considered part of the length of need for the run of guardrail; see AASHTO *Roadside Design Guide* and the *SCDOT Standard Drawings*. Do not include this portion of guardrail end treatment when estimating the length of guardrail run for payment.

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Example 20.7-1

Given: Guardrail Length of Need for 2H:1V slope = 1500 feet
Two-lane Arterial

Problem: Compute the estimated quantities for installing the run of guardrail.

Solution:

1. For a two-lane arterial, two Type T (TL-3) guardrail end treatments are required. The pay quantity is each.

2. The pay limits for a Type T, TL-3 end treatment is 50 feet. However, only 37.5 feet of the end treatment can be considered as part of the guardrail length of need. To estimate the pay quantity for the length of guardrail required, subtract the portion of the end treatment that can be used as part of the length of need from the overall length of need and ensure this length of guardrail is divisible by 12.5 feet.

$$1{,}500\text{ FT} - (2 \times 37.5\text{ ft}) = 1{,}425\text{ FT}$$

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20.7.2 Underground Storage Tanks

There are three pay items for underground storage tanks:

- Removal and Disposal of Tank Contents,
- Removal and Disposal of Low Level Contaminated Soil, and
- Removal and Disposal of High Level Contaminated Soil.

For additional information on underground storage tanks, see Section 22.2.4.2.3.

20.7.3 <u>Driveways</u>

20.7.3.1 General

Only show driveway changes, additions or new driveways on the plans if locations have been pre-determined by the Program Manager. Existing driveways need to be shown. The maintenance stone specified in Section 20.7.3.2 will serve the base of the driveway. For each driveway, specify an asphalt overlay of 400 pounds per square yard using the surface course material for the mainline pavement. Include these quantities in the inclusions on the General Construction Note Sheet. Other alternatives may be used as discussed at the Design Field Review.

Driveways should be constructed in accordance with the *SCDOT Standard Drawings* and *SCDOT Access and Roadside Management Standards*.

Where projects require sidewalk construction, or improvements are proposed in areas with sidewalk, driveways are constructed of concrete and appropriate quantities are included in the plans.

Where concrete sidewalk is called for on a project with driveways, a quantity for Concrete Driveway at a uniform depth of 6 inches is included for all driveways. Deduct this quantity from the sidewalk quantity. See the *SDOT Standard Drawings* for details.

20.7.3.2 Maintenance of Roadway and Driveways During Construction

Where material is needed to maintain traffic on the roadway or on driveways during construction, include the pay item Maintenance Stone (TON). Materials used to maintain driveways during construction must meet the requirements of the *SCDOT Standard Specifications* and are placed in the inclusions. For driveways, use 25 tons of Maintenance Stone per 100 square yards. Additional quantities may be requested by the District at the Design Field Review.

20.7.3.3 Pipe Under Driveways

The following pipe quantities are required for driveways during construction:

- a minimum length of 24 feet of pipe is provided for each standard driveway;
- additional pipe of various sizes may be shown in the plans as an inclusion item. This is determined during the Design Field Review; and
- a change in pipe length is determined by the Resident Construction Engineer.

In all instances, show proposed drive pipe sizes and lengths on the plans.

20.7.4 <u>Aggregate Underdrain</u>

A quantity of aggregate underdrain is calculated and included for pipe underdrains of all sizes in accordance with the *SCDOT Standard Specifications*. For this purpose, fine aggregate (FA-12 or FA-13) is specified and used to backfill the trench above the minimum amount of required course aggregate placed with the pipe underdrain.

Calculate fine aggregate in accordance with the rates listed below:

- 4 inches – 100 cubic yards per 1,000 linear feet of Pipe Underdrain
- 6 inches – 130 cubic yards per 1,000 linear feet of Pipe Underdrain
- 8 inches – 150 cubic yards per 1,000 linear feet of Pipe Underdrain

20.7.5 Pedestrian Ramps

Use the item Removal and Disposal of Existing Pavement for locations where existing sidewalk or pavement must be removed in order to construct a new pedestrian ramp; see Section 20.2.8.

Calculate the number of pedestrian ramps that will use newly placed concrete (n). The value for n should include all new ramp locations and all locations where existing sidewalk will be removed (r). Multiply n by the average area of pedestrian ramp construction (estimated as 25 SY) to determine the plan quantity for pedestrian ramp construction. The Pedestrian Ramp Construction quantity includes all work necessary to construct the ramp excluding Detectable Warning Material.

Plan Quantity for Pedestrian Ramp Construction = $n \times 25$ SY

Include a quantity for Detectable Warning Material for all locations with Pedestrian Ramp Construction. The quantity of Detectable Warning Material in SF should be equal to about half the SY of Pedestrian Ramp Construction used. Calculate the plan quantity for Detectable Warning Material by dividing the Pedestrian Ramp Construction quantity by 2.

Plan Quantity for Detectable Warning Material = $(n \times 25)/2$ SF

Note that the pay item Surface Applied Detectable Warning will only be present on projects with crossings through raised medians or when existing ramps only need the application of detectable warning material. Do not calculate this quantity for any ramp included in the n quantity above.

Calculate the number of pedestrian crossings into refuge islands (m) + any ramps where only warning surface will be applied. Multiply m by 10 SF to determine the plan quantity for Surface Applied Detectable Warnings.

Plan Quantity for Surface Applied Detectable Warning = $m \times 10$ SF

If present, measure the concrete median as directed in the *SCDOT Standard Specifications*.

20.7.6 Fences

On all highway projects existing fences within the construction area or right of way that are constructed of materials other than standard fence materials (i.e., chain link or woven wire) are set up as Moving Items, to be removed or reconstructed, as appropriate. These fences typically consist of brick, concrete block, rail, wrought iron or another type of decorative material.

Where re-setting fences is necessary and the type of fence is chain link, specify it as Reset Chain-Link Fence while Reset Fence is adequate for all other types. Reset Fence is shown in linear feet in the General Construction Notes and on the Roadway Summary of Estimated Quantity Sheet. New Fence is shown in linear feet on the Moving Item Sheet and is not included on the General

Construction Note Sheet. All new and reset fence quantities are provided by the Rights of Way Office.

20.7.7 <u>Shrubs and Plants</u>

It is important that the number of shrubs and plants being relocated is set up as Moving Items and is itemized as accurately as practical. The Rights of Way Office will provide the designer with the quantities for plant and shrubs that are to be handled as Moving Items.

20.7.8 <u>Seeding, Mulching and Sodding</u>

The Design Field Review will determine the seeding types, mulched or unmulched and/or the need for sodding. Estimate the quantity of permanent vegetation assuming 3.0 MSY per mile for local roads and 4.7 MSY per mile for State roads. If permanent vegetation will be provided, do not estimate quantities for seeding, fertilizer, lime and nitrogen. They are included in the bid price for permanent vegetation. When appropriate, include a bid item for seeding when it is expected that the contractor will disturb an area that would subsequently require reseeding.

Do not include quantities for fertilizer and lime when specifying any type of sprigging or sodding. Fertilizer and lime are included in the bid price for sprigging or sodding. Fertilizer and lime will be applied in accordance with the most current version of SCDOT Supplemental Technical Specification SC-M-810.

20.7.8.1 Mulch

Mulch is required for all temporary and permanent cover applications. See SCDOT Supplemental Technical Specification SC-M-815 for guidance.

It is recommended that the bid quantity in acres for all Hydraulic Erosion Control Products be equal to twice the calculated acreage to be mulched. This will allow for re-mulching areas due to adverse weather conditions, re-disturbance of soil, etc.

The designer can weigh the options of using multiple types of mulches or combining items when there is just a small amount of one type of mulch and a much larger amount of a higher performance mulch.

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<u>Example 20.7-2</u>

Given: Straw or Hay Mulch with Tackifier = 2.346 acres
HECP Type 3 = 0.398 acres
HECP Type 4 = 5.443 acres

The designer could choose to combine two of the three options, but this option is not required.

Straw or Hay Mulch with Tackifier = 2.346 acres
HECP Type 4 = 5.841 acres

This simplifies the construction operation in that the contractor only supplies two types of mulch products. The impact of the cost is minimal due to the small quantity of the Type 3 mulch. Note that the Type 4 mulch is the highest performance mulch. When choosing to combine mulch quantities, always choose the higher performance mulch to replace the lower performance mulch.

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20.7.8.2 Agricultural Granular Lime

Estimate the quantity of agricultural granular lime using the rate of 2000 pounds/acre of permanent cover.

20.7.8.3 Agricultural Granular Fertilizer

Unless a specific blend of granular fertilizer is selected during the Design Field Review or based on experience, assume a 10-10-10 mix to calculate the estimated bid quantities for the three components of granular fertilizer. The first number represents the percentage of nitrogen, the second number represents the percentage of phosphoric acid and the third number represents the percentage of potash (N- Ph. Acid-Pot.). The numbers are percentages; divide the numbers by 100 to calculate the amount of each item required.

Calculate the quantity fertilizer by using an application rate of 1000 pounds/acre.

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Example 20.7-3

Given: Permanent Cover = 3,500 acres

Problem: The Resident Construction Engineer recommends a 15-5-25 mix of fertilizer. Calculate the estimated quantity.

Solution:

1. Fertilizer (Nitrogen) = 0.15 × 1000 pounds/acre × 3.500 acres = 525 pounds
2. Fertilizer (Phosphoric Acid) = 0.05 × 1000 pounds/acre × 3.500 acre = 175 pounds
3. Fertilizer (Potash) = 0.25 × 1000 pounds/acre × 3.500 acres = 875 pounds

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20.7.8.4 Compost

Compost can be used in locations where there is little or no topsoil or where a soil analysis shows that the seedbed is excessively nutrient deficient. This information is generally unknown during the project development phase. See the latest edition of the SCDOT Supplemental Technical Specification SC-M-815 for guidance on compost.

The designer should request input from the Resident Construction Engineer during the Design Field Review to determine if this bid item is necessary and estimate the quantity based on past experience. Compost is measured in cubic yards. To determine the volume needed, estimate the surface area and use a depth of 2 inches.

20.7.8.5 Watering

To establish permanent cover, water may be required. Calculate a quantity using the ration of 27,150 gallons per acre-inch of water. Select the quantity of watering using Figure 20.7-A based on the area of permanent cover. See the latest edition of the SCDOT Supplemental Technical Specification SC-M-810 for additional guidance.

Area of Permanent Cover	Quantity of Water
less than ½ acre	None
½ acre to 5 acres	54,300 gallons
Greater than 5 acres to 20 acres	135,750 gallons
Greater than 20 acres	271,500 gallons

QUANTITY OF WATER
Figure 20.7-A

20.7.8.6 Mowing

Include Mowing when permanent or temporary cover is specified. Use a quantity equal to twice the sum of the permanent and temporary cover.

20.7.9 Ditch Paving and Flumes at Vertical Parapet on Bridges

The designer should use bridge approach concrete curb and gutter with flume along with a paved shoulder area on all bridge ends. See the *SCDOT Standard Drawings* for related pay items.

Liquid asphalt binder is included in hot-mix surface course for ditch paving. Use the same pavement design used on the roadway for shoulder.

20.7.10 Right-of-Way Plats and Monuments

Include right-of-way plats and monuments bid items on all projects that require new right of way. See the applicable design memorandum and the *SCDOT Standard Drawings* for construction and installation details.

Include the pay items for reinforced concrete markers on Interstate routes and other controlled access facilities and rebar and cap markers on all other routes.

20.7.11 Brick Masonry and Reinforced Brick Masonry Walls

Calculate quantity for Brick Masonry and Reinforced Brick Masonry Walls based on volume of wall (Length × Width × Height).

20.7.12 Quality Control by Contractor

The decision to use a quality control pay item will be made during the Design Field Review by the District Construction Office. Include this pay item on the Summary of Estimated Quantities Sheet.

20.7.13 Stabilized Construction Entrance

Any construction project that may have one or more equipment entrances over the duration of the project to the construction site from a paved roadway will require the installation of a stabilized construction entrance. The designer should provide enough quantities for at least two entrances. Projects may have more than one active entrance at a time. Construction entrances may move to different locations as construction stages change.

In order to estimate the quantity of stabilized construction entrance on a project, the designer will provide the number of square yards for one stabilized construction entrance for every 2600 feet of mainline with a minimum of two per project. For estimation purposes, use 275 square yards per entrance. If the project is not disposed to having an entrance location where there will be concentrated construction equipment traffic, then quantities for a stabilized construction entrance will not be required. If the designer cannot determine if a project should have quantities for stabilized construction entrance, it is recommended to provide the pay item and quantities as discussed above.

A stabilized construction entrance is not to be used where normal non-construction traffic will travel.

SPACER PAGE

20.8 MATHEMATICAL FORMULA

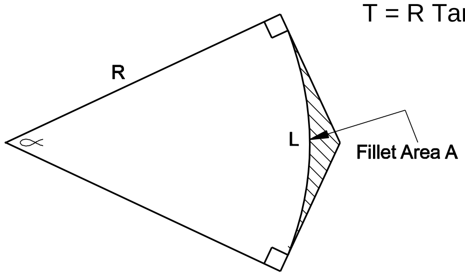
The following figures present various mathematical relationships to assist the designer with estimating tasks:

- Figure 20.8-A — ASTM STANDARD REINFORCING BARS
- Figure 20.8-B — AREA OF FILLETS, APRONS AND INTERSECTION APPROACHES
- Figure 20.8-C — PROPERTIES OF THE CIRCLE
- Figure 20.8-D — TRIGONOMETRIC FORMULAS OF TRIANGLES
- Figure 20.8-E — AREAS OF PLANE FIGURES

ASTM Standard Reinforcing Bars			
Bar Size Designation	**Area (Square Inches)**	**Weight (Pounds/Foot)**	**Diameter (Inches)**
#3	0.11	0.376	0.375
#4	0.20	0.668	0.500
#5	0.31	1.043	0.625
#6	0.44	1.502	0.750
#7	0.60	2.044	0.875
#8	0.79	2.670	1.00
#9	1.00	3.400	1.128
#10	1.27	4.303	1.270
#11	1.56	5.313	1.410
#14	2.25	7.650	1.693
#18	4.00	13.600	2.257

ASTM STANDARD REINFORCING BARS
Figure 20.8-A

$$T = R \text{ Tan } \frac{\alpha}{2}$$



ESTIMATING FILLETS & RETURN

Fillet Area

$$\text{Area A} = 2 \times \frac{1}{2} \times R \times R \text{ Tan } \frac{\alpha}{2} - \pi R^2 \frac{\alpha}{360°}$$

$$= R^2 \left[\text{Tan} \frac{\alpha}{2} - (0.008727 \times \alpha) \right]$$

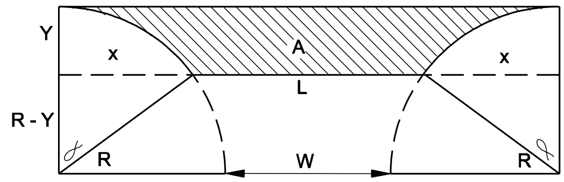
Area 90° Fillet = $0.2146 \times R^2$

Length of Return

$$L = 2\pi R \times \frac{\alpha}{360°}$$

$$= 0.01745 \times R \times \alpha$$

Length of 90° Return = $1.5708 \times R$

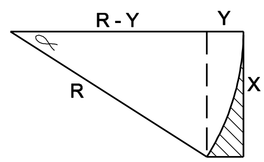


ESTIMATING AREA 90° APRON

$L = (2R + W) - 2X$ $\quad\quad$ $\text{Cos } \alpha = \dfrac{R - Y}{R}$

$X = \sqrt{2RY} - Y^2$ $\quad\quad$ A = Area

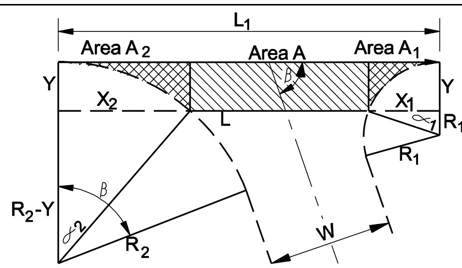
$A = (2R + W)Y + X(R - Y) - 0.01745 \ R^2\alpha$



A = Area

$$A = XY - \left[\pi R^2 \frac{\alpha}{360°} - \frac{1}{2} \times (R - Y)\right]$$

$$= XY + \frac{X(R - Y)}{2} - 0.08727 \ R^2 \alpha$$



ESTIMATING AREA APRON OTHER THAN 90°

$\text{Cos } \alpha_1 = \dfrac{R_1 - Y}{R_1}$ $\quad\quad$ $\text{Cos } \alpha_2 = \dfrac{R_2 - Y}{R_2}$

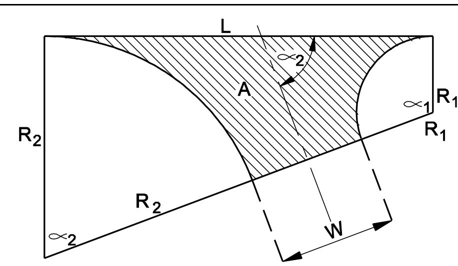
$X_1 = \sqrt{2R_1 Y - Y^2}$ $\quad\quad$ $X_2 = \sqrt{2R_2 Y - Y^2}$

$L_1 = (R_2 - R_1) \text{ Tan } \beta$ $\quad\quad$ $L = L_1 - (X_1 + X_2)$

$A = LY$

$$A_1 = X_1 Y + \frac{X_1(R_1 - Y)}{2} - 0.008727 R_1^2 \alpha_1$$

$$A_2 = X_2 Y + \frac{X_2(R_2 - Y)}{2} - 0.008727 R_2^2 \alpha_2$$



ESTIMATING AREA APPROACH OTHER THAN 90°

$\alpha_1 = 180° - \alpha_2$

$L = (R_2 - R_1) \text{ Tan } \alpha_2$

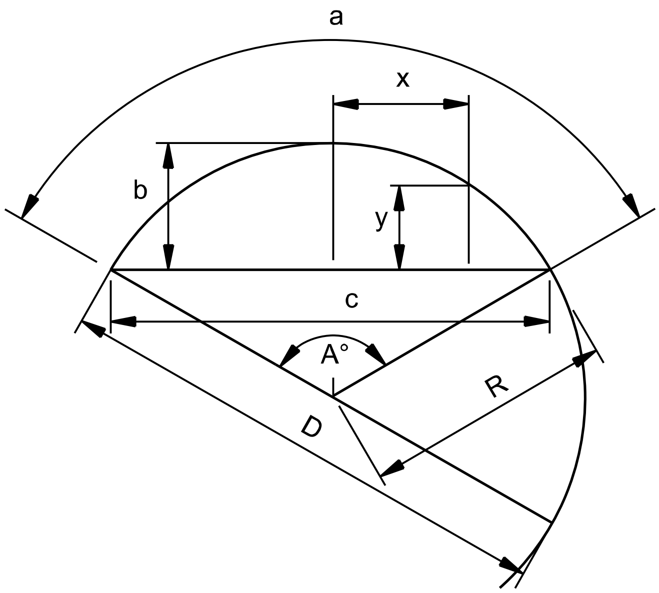
$$\text{Area A} = \frac{(R_1 + R_2)L}{2} - 0.008727(R_1^2 \alpha_1 + R_2^2 \alpha_2)$$

NOTES:

$\pi = 3.1416$ $\quad\quad$ $\dfrac{\pi}{180} = 0.01745$

$\dfrac{\pi}{2} = 1.5708$ $\quad\quad$ $\dfrac{\pi}{360} = 0.008727$

AREA OF FILLETS, APRONS AND INTERSECTION APPROACHES
Figure 20.8-B



Circumference = $2\pi R = \pi D$

Diameter = 0.31831 circumference

Area = πR^2

Arc $a = \dfrac{\pi R\, A°}{180°} = 0.017453\ R\ A°$

Angle $A° = \dfrac{180°\, a}{\pi R} = 57.29578\ \dfrac{a}{R}$

Radius $R = \dfrac{4\,b^2 + c^2}{8\,b}$

Chord $c = 2\sqrt{2\,bR - b^2} = 2\,R\sin\dfrac{A}{2}$

Rise $b = R - \tfrac{1}{2}\sqrt{4\,R^2 - c^2} = \dfrac{c}{2}\tan\dfrac{A}{4}$

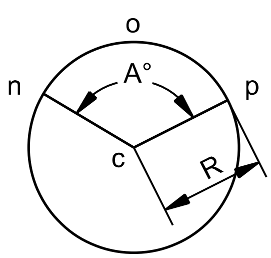
$= 2\,R\sin^2\dfrac{A}{4} = R + y - \sqrt{R^2 - x^2}$

$y = b - R + \sqrt{R^2 - x^2}$

$x = \sqrt{R^2 - (R + y - b)^2}$

Diameter of circle of equal periphery as square	=	1.27324 side of square
Side of square of equal periphery as circle	=	0.78540 diameter of circle
Diameter of circle circumscribed about square	=	1.41421 side of square
Side of square inscribed in circle	=	0.70711 diameter of circle

CIRCULAR SECTOR



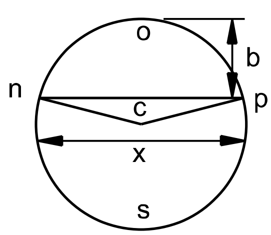
R = radius of circle   A° = angle ncp in degrees

Area of Sector ncpc = ½ (length of arc nop × R)

$= \text{Area of Circle} \times \dfrac{A°}{360}$

$= 0.0087268 \times R^2 \times A°$

CIRCULAR SEGMENT



R = radius of circle   x = chord   b = rise

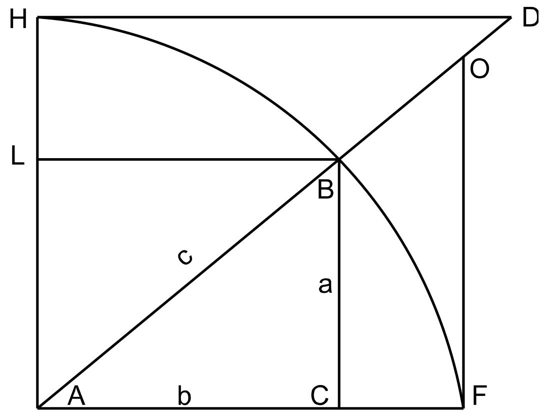
Area of Segment nop = Area of Sector ncpo – Area of triangle ncp

$= \dfrac{(\text{Length of arc nop} \times r) - x\,(r - b)}{2}$

Area of Segment nsp = Area of Circle – Area of Segment nop

PROPERTIES OF THE CIRCLE
Figure 20.8-C

TRIGONOMETRIC FUNCTIONS



Radius AF $= 1$

$$= \sin^2 A + \cos^2 A = \sin A \operatorname{cosec} A$$

$$= \cos A \sec A = \tan A \cot A$$

$$\text{Sine } A = \frac{\cos A}{\cot A} = \frac{1}{\operatorname{cosec} A} = \cos A \tan A = \sqrt{1-\cos^2 A} = BC$$

$$\text{Cosine } A = \frac{\sin A}{\tan A} = \frac{1}{\sec A} = \sin A \cot A = \sqrt{1-\sin^2 A} = AC$$

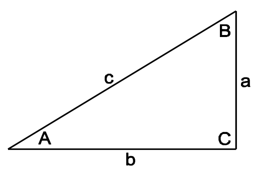
$$\text{Tangent } A = \frac{\sin A}{\cos A} = \frac{1}{\cot A} = \sin A \sec A = FD$$

$$\text{Cotangent } A = \frac{\cos A}{\sin A} = \frac{1}{\tan A} = \cos A \operatorname{cosec} A = HG$$

$$\text{Secant } A = \frac{\tan A}{\sin A} = \frac{1}{\cos A} = AD$$

$$\text{Cosecant } A = \frac{\cot A}{\cos A} = \frac{1}{\sin A} = AG$$

RIGHT ANGLED TRIANGLES



$$a^2 = c^2 - b^2$$
$$b^2 = c^2 - a^2$$
$$c^2 = a^2 - b^2$$

Known	Required					
	A	B	a	b	c	Area
a, b	$\tan A = \frac{a}{b}$	$\tan B = \frac{b}{a}$			$\sqrt{a^2 + b^2}$	$\frac{ab}{2}$
a, c	$\sin A = \frac{a}{c}$	$\cos B = \frac{a}{c}$		$\sqrt{c^2 - a^2}$		$\frac{a\sqrt{c^2 - a^2}}{2}$
A, a		90° – A		a cot A	$\frac{a}{\sin A}$	$\frac{a^2 \cot A}{2}$
A, b		90° – A	b tan A		$\frac{b}{\cos A}$	$\frac{b^2 \tan A}{2}$
A, c		90° – A	c sin A	c cos A		$\frac{c^2 \sin 2A}{4}$

TRIGONOMETRIC FORMULAS OF TRIANGLES
Figure 20.8-D

OBLIQUE ANGLED TRIANGLES

$$s = \frac{a + b + c}{2}$$

$$K = \sqrt{\frac{(s-a)(a-b)(s-c)}{s}}$$

$a^2 = b^2 + c^2 - 2\ bc\ \cos A$

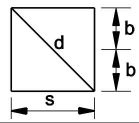
$b^2 = a^2 + c^2 - 2\ ac\ \cos B$

$c^2 = a^2 + c^2 - 2\ ab\ \cos C$

Known	Required					
	A	B	C	b	c	Area
a, b, c	$\tan\frac{1}{2}A = \frac{K}{s-a}$	$\tan\frac{1}{2}B = \frac{K}{s-b}$	$\tan\frac{1}{2}C = \frac{K}{s-c}$			$\sqrt{s(s-a)(s-b)s-c}$
a, A, B			180°–(A+B)	$\frac{a \sin B}{\sin A}$	$\frac{a \sin C}{\sin A}$	
a, b, A		$\sin B = \frac{b \sin A}{a}$			$\frac{b \sin C}{\sin B}$	
a, b, C	$\tan A = \frac{a \sin C}{b - a \cos C}$				$\sqrt{a^2 + b^2 - 2ab\cos C}$	$\frac{ab \sin C}{2}$

TRIGONOMETRIC FORMULAS OF TRIANGLES
Figure 20.8-D
(Continued)

Square



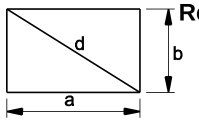
Diagonal = d = $s\sqrt{2}$

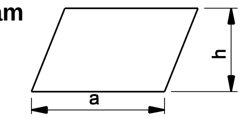
Area = $s^2 = 4b^2 = 0.5\ d^2$

Example: s = 6; b = 3; Area = $(6)^2 = 36$

d = 6 x 1.414 = 8.484

Rectangle and Parallelogram



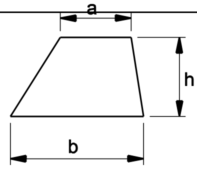


Area = ab or $b\sqrt{d^2 - b^2}$

Example: a = 6; b = 3

Area = 3 x 6 = 18

Trapezoid

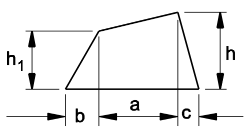


Area = $\frac{1}{2}h\left(a+b\right)$

Example: a = 2; b = 4; h = 3

Area = $\frac{1}{2}\times 3\left(2+4\right) = 9$

Trapezium

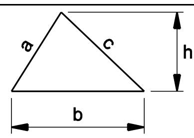


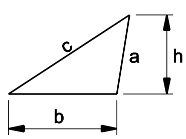
Area = $\frac{1}{2}\left[a\left(h+h_1\right)+bh_1+ch\right]$

Example: a = 4; b = 2; c = 2; h = 3; $h_1 = 2$

Area = $\frac{1}{2}\left[4\left(3+2\right)+\left(2\times 2\right)+\left(2\times 3\right)\right] = 15$

Triangles





Formulas apply to both figures.

Area = $\frac{1}{2}\ bh$

Example: h = 3; b = 5

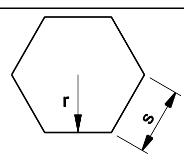
Area = $\frac{1}{2}\left(3\times 5\right) = 7.5$

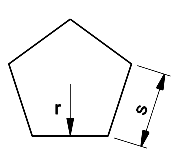
Area = $\sqrt{s\left(s-a\right)\left(s-b\right)\left(s-c\right)}$ where: $s = \frac{a+b+c}{2}$

Example: a = 2; b = 3; c = 4

$s = \frac{2+3+4}{2} = 4.5$; Area = $\sqrt{4.5\left(4.5-2\right)\left(4.5-3\right)\left(4.5-4\right)} = 2.9$

Regular Polygons





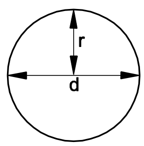
$$\text{Area}\left\langle\begin{array}{l} 5\text{ sides} = 1.720477\ s^2 = 3.63271\ r^2 \\ 6\text{ sides} = 2.598150\ s^2 = 3.46410\ r^2 \\ 7\text{ sides} = 3.633875\ s^2 = 3.37101\ r^2 \\ 8\text{ sides} = 4.828427\ s^2 = 3.31368\ r^2 \\ 9\text{ sides} = 6.181875\ s^2 = 3.27573\ r^2 \\ 10\text{ sides} = 7.694250\ s^2 = 3.24920\ r^2 \\ 11\text{ sides} = 9.365675\ s^2 = 3.22993\ r^2 \\ 12\text{ sides} = 11.196300\ s^2 = 3.21539\ r^2 \end{array}\right.$$

n = number of sides; r = short radius; s = length of side; R = long radius.

Area = $\frac{n}{4}s^2\cot\frac{180°}{n} = \frac{n}{2}R^2\sin\frac{360°}{n} = nr^2\tan\frac{180°}{n}$

AREA OF PLANE FIGURES
Figure 20.8-E

Circle



$\pi = 3.1416$; A = area; d = diameter

p = circumference or periphery; r = radius

$p = \pi d = 3.1416d$ $\qquad$ $p = 2\sqrt{\pi A} = 3.54\sqrt{A}$

$p = 2\pi r = 6.2832r$ $\qquad$ $p = \frac{2A}{r} = \frac{4A}{d}$

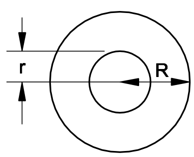
$d = \frac{p}{\pi} = \frac{p}{3.1416}$ $\qquad$ $d = 2\sqrt{\frac{A}{\pi}} = 1.128\sqrt{A}$

$r = \frac{p}{2\pi} = \frac{p}{6.2832}$ $\qquad$ $r = \sqrt{\frac{A}{\pi}} = 0.564\sqrt{A}$

$A = \frac{\pi d^2}{4} = 0.7854d^2$ $\qquad$ $A = \frac{p^2}{4\pi} = \frac{p^2}{12.57}$

$A = \pi r^2 = 3.1416r^2$ $\qquad$ $A = \frac{pr}{2} = \frac{pd}{4}$

Circular Ring



Area = $\pi(R^2 - r^2) = 3.1416\ (R^2 - r^2)$

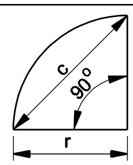
Area = $0.7854\ (D^2 - d^2) = 0.7854(D - d)(D + d)$

Area = difference in areas between the inner and outer circles.

Example: R = 4; r = 2

Area = $3.1416(4^2 - 2^2) = 37.6992$

Quadrant

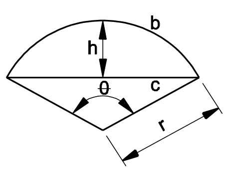


Area = $\frac{\pi r^2}{4} = 0.7854\ r^2 = 0.3927c^2$

Example: r = 3; c = chord

Area = $0.7851 \times 3^2 = 7.0686$

Segment



b = length of arc; θ = angle in degrees; c = chord = $\sqrt{4(2hr - h^2)}$

Area = $\frac{1}{2}\left[br - c(r - h)\right] = \pi r^2 \frac{\theta}{360} - \frac{c(r-h)}{2} p$

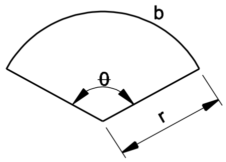
When θ is greater than 180° then $\frac{c}{2}$ x difference between r and h is added to the fraction $\frac{\pi r^2 \theta}{360}$

Example: r = 3; θ = 120°; h = 1.5

Area = $3.1416 \times 3^2 \times \frac{120}{360} - \frac{5.196\,(3 - 1.5)}{2} = 5.5278$

AREA OF PLANE FIGURES
Figure 20.8-E
(Continued)

Sector



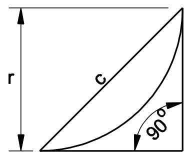
$$\text{Area} = \frac{br}{2} = \pi r^2 \frac{\theta}{360°}$$

θ = angle in degrees; b = length of arc

Example: r = 3; θ = 120°

$$\text{Area} = 3.1416 \times 3^2 \times \frac{120}{360} = 9.4248$$

Spandrel

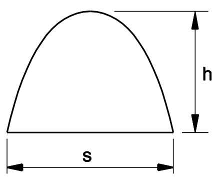


Area = 0.2146 r^2 = 0.1073 c^2

Example: r = 3

Area = 0.2146 x 3^2 = 1.9314

Parabola



l = length of curved line = periphery – s

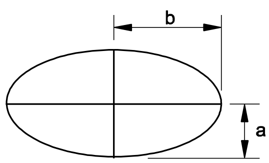
$$l = \frac{s^2}{8h}\left[\sqrt{c(1+c)} + 2.0326 \text{ x } \log\left(\sqrt{c} + \sqrt{1+c}\right)\right] \text{ where } c = \left(\frac{4h}{s}\right)^2$$

$$\text{Area} = \frac{2}{3}sh$$

Example: s = 3; h = 4

$$\text{Area} = \frac{2}{3} \times 3 \times 4 = 8$$

Ellipse



Area = πab = 3.1416ab

$$\text{Circumference} = 2\pi\sqrt{\frac{a^2 + b^2}{2}} \text{ (close approximation)}$$

Example: a = 3; b = 4.

Area = 3.1416 x 3 x 4 = 37.6992

$$\text{Circumference} = 2 \text{ x } 3.1416\sqrt{\frac{(3)^2 + (4)^2}{2}} = 6.2832 \text{ x } 3.5355 = 22.21$$

AREA OF PLANE FIGURES
Figure 20.8-E
(Continued)

20.9 REFERENCES

1. *Standard Specifications for Highway Construction*, SCDOT, Current Edition.
2. *Supplemental Technical Specifications*, SCDOT, Current Edition.
3. *Book of Standard Drawings for Road Construction*, SCDOT, Current Edition.
4. *Roadside Design Guide*, AASHTO, 2011.

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Chapter 21

PROCEDURES FOR HIGHWAY PLANS PREPARATION

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Section **Page**

Chapter 21
PROCEDURES FOR HIGHWAY PLANS PREPARATION

This chapter provides an overview of the design team's responsibilities for the development of SCDOT highway plans. Chapter 22 "Plan Sheets Preparation" provides detailed guidelines on plan sheet content.

21.1 PRELIMINARY ENGINEERING

The following sections identify several procedures, but not all, the designer should consider during the Preliminary Engineering design stage. For further guidance, see the Preconstruction Project Development Process on SCDOT's internet site.

21.1.1 Survey Requests

The determination of the appropriate point in the highway development process to begin field surveys will depend on a variety of factors. The main consideration is the complexity of the project. A new freeway or roadway on a new location should be developed to the point that the preliminary location and design studies have been completed before the field survey work begins. For guidance on when a survey should be requested, see the Preconstruction Project Development Process.

Surveys are requested from the Surveys Office. The designer must include with the request the appropriate roadway and bridge plans, USGS Maps and other support data. The designer may use topographic maps or other survey source materials to develop the proposed survey centerline. For smaller projects, the designer may only be required to submit the county map showing the project's location to the Survey Office to initiate the field survey. Survey crews will collect the necessary information (e.g., tax maps, plats, deeds, easement records) to complete the data research for the survey needs. For more information on survey requirements, see the *SCDOT Survey Manual.*

21.1.2 Survey Data

All topographic data (e.g., fence corner, power pole, mailbox, 10-inch oak) will be noted on the plans via station and offset from the survey centerline. Station and offset may be shown on the plans 90 degrees to the survey centerline and left and right of the centerline or on a separate sheet. If there is an extensive amount of topographic information, it can be shown in tabular form.

Developing base sheets through aerial mapping procedures will require surveyed monument points in the field or stringently controlled traverses wherein identifiable photographic points are coordinated. Even though the resulting topographic mapping data are very accurate, selective station and offset text for all topographic points is required. Examples include large stately trees, house/porch corners, wells and any features not identified by the aerial mapping, all of which may lie within a few feet of the new right of way.

21.1.3 Project Location Controls

Initial design considerations involve the establishment of the graphical location of the project and its termini. The roadway designer should refine the schematic roadway to a cost effective geometric layout. Although straight-line links are the shortest distance between project termini or intermediate points, they may not be the most cost effective due to terrain considerations and/or possible conflicts with environmentally sensitive areas. The roadway designer should identify the location of all critical controls. To assist with the NEPA process, note any alignment changes to minimize impacts to natural resources. For guidance on alignment controls, see Chapter 5 "Horizontal Alignment" and Chapter 6 "Vertical Alignment."

21.1.4 Stationing Alignments

Upon completion of a graphical alignment, the designer should set the stationing of the alignment to provide a method of identification of specific locations along the alignment and as a method of coordinating locations between plan and profile views. If there are no previous stations established in record plans, the designer may assign an initial beginning stationing number. It is recommended that the assigned beginning number be other than zero and preferably be an exponent of 10 (e.g., 100, 1000, 10,000). Avoiding the use of zero as a beginning station will reduce the potential of negative stationing should the project be extended in the back stationing direction.

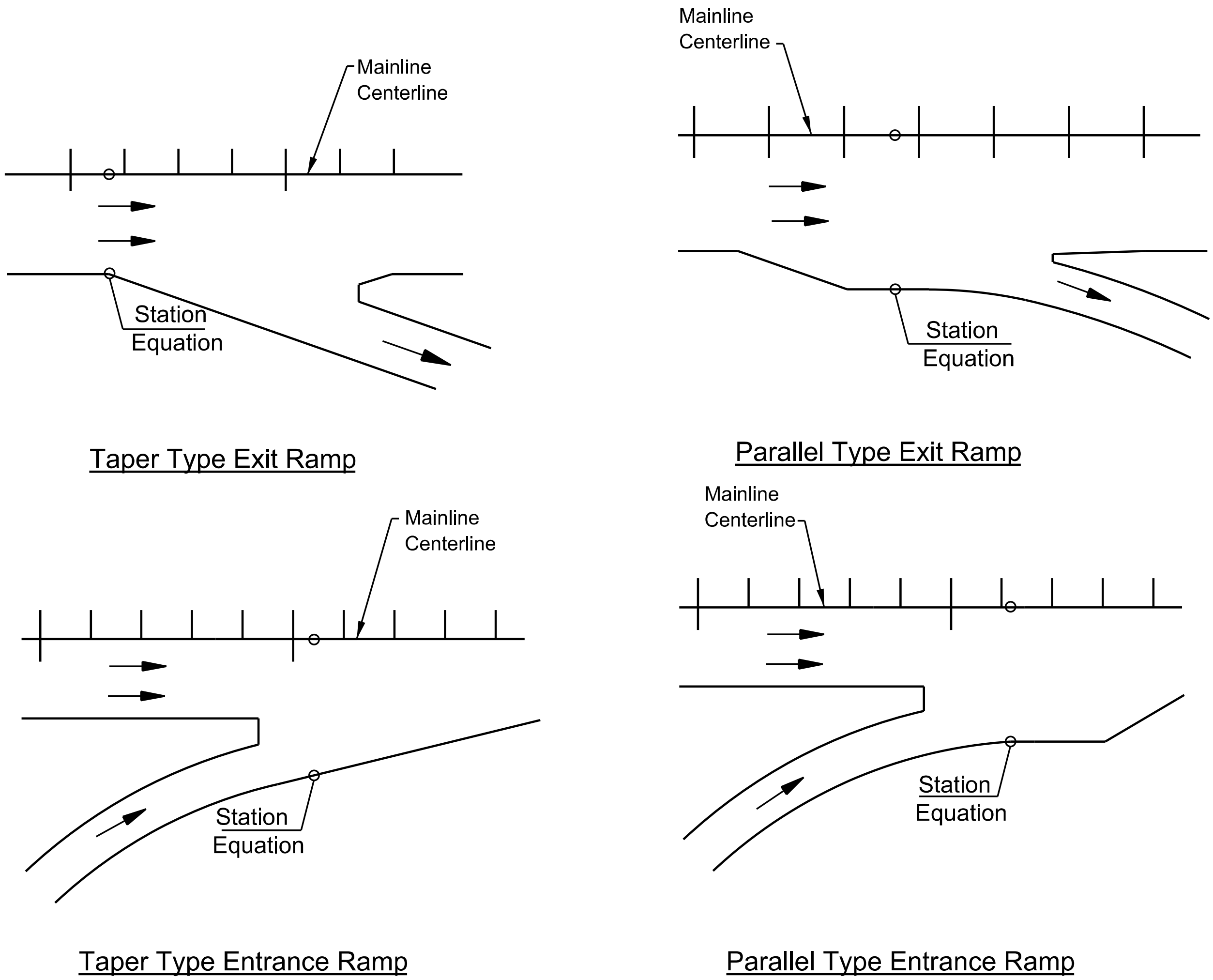
Stationing is defined as a length of 100 feet (i.e., STA 10 is actually 10 one hundred foot lengths and is written as 10+00). In the stationing of an alignment, standard practice is to begin the stationing at the south or west termini of the project and to increase stationing numbers from south to north or from west to east. This is consistent with conventional reading from left to right for west to east stationing and from left to right for south to north stationing.

Stationing on ramps should increase in the same direction as the mainline. The beginning station should be common with the mainline station from which the ramp diverges (e.g., STA. 439 + 16.21, Line B is 42 feet Lt. STA. 439 + 16.21, I-85). When stationing the ramp centerline, establish the proper equality as depicted on Figure 21.1-A and station the ramp forward or backward as required to maintain ramp stationing in the same increasing direction as the mainline. At an interchange, station the ramps and connecting roads to be consistent with the mainline facility.

21.1.5 Line and Grade Design Field Review

During the early development of a project, the geometric design is selected and placed on the plans. The project templates are shown on the cross sections providing the footprint of the proposed roadway. Prior to initiating the hydraulic design, a field review will be held to verify the geometry of a new design.

A Design Field Review will be held to determine if the vertical and horizontal alignment, as indicated on the plans, meets the scope of the project. The review team should closely examine the existing conditions of the project alignment and make modifications as necessary. The team will ensure that the design indicated will become the final geometric design used on the project. This is when the project is first reviewed for the controlling design criteria; see Section 3.2.



STATION EQUATIONS FOR VARIOUS RAMPS

Figure 21.1-A

The review team should be composed of those with specific interest in the geometry of the roadway. A review of the other features of the project (e.g., hydrology and erosion control) will be held at a later time. The team should include, but is not limited to, representatives from Project Management, District Engineering, Utility Coordination, Right of Way, Environmental and Roadway Design. It is the responsibility of the roadway designer to set-up the Design Field Review and lead the team through the project on location.

The designer needs to ensure that the Design Field Review Title Sheet is used for all Design Field Reviews (DFR). The applicable Title Sheet is provided on the SCDOT's internet site. Prior to the field review, the DFR plans will be an electronically created PDF file submitted to Engineering Reproductions Services (ERS) made available through the Plans Library. When notifying the review team, the Design Group will advise that the plans are available in the Plans Library. Upon returning from the DFR, plans will be scanned by ERS making them available through the Plans Library.

21.1.6 Right of Way

21.1.6.1 Right of Way Control

The Design Field Review Plans should show the approximate new right-of-way line and control-of-access lines, if applicable. Place the control-of-access lines just inside the right-of-way line. If no control-of-access lines are shown, then it is understood that access to the highway by the adjoining property owners is permitted.

21.1.6.2 Right of Way Plans

The Right of Way Plans should conform to the standard practices and procedures presented in Chapter 12 "Right of Way." The right-of-way requirements are determined by plotting the construction limits. The designer typically establishes a new right-of-way line outside of the construction limits. The locations of new right-of-way limits should be evaluated in-depth at the Design Field Review and adjusted as required by conditions. In some cases (e.g., congested urban areas), the right-of-way line is set a minimum of 0.5 feet from the back edge of the sidewalk; slopes that extend beyond these limits are acquired by slope permission agreements obtained by the Rights of Way Office.

For major projects, the completed Right of Way Plans should depict the entire parcel of property affected by the proposed acquisition. This may be achieved on the Plan and Profile Sheets, supplemental reduced scale plots of entire property boundaries on separate sheets, or a combination of the above. Accurate representation of existing development is essential for proper appraisals and negotiations of acquiring additional right of way.

21.1.7 Typical Sections

See the Advanced Project Planning Report (APPR) or Corridor Feasibility Study for typical section information. If the APPR is available, coordinate with the Project Development Team to establish typical sections. The Typical Sections at this stage will only provide enough information to reflect the project's roadway elements. For example, a detailed dimensioning of the pavement composition is not necessary at this stage. For more information on Typical Sections, see Section 22.2.5.

21.1.8 Pavement Design

Pavement designs are requested by the roadway designer when required. Request a classification count with the traffic loading data from the traffic designer so that the appropriate pavement design can be determined. As the designer develops typical sections for the project, the most current pavement design available will be used to establish the materials and rates shown on the Typical Section Sheet. The accuracy of the pavement design is very important to the success of its implementation during the construction phase and the life of the roadway. Therefore, when the Typical Section Sheet is prepared, a plan size original is to be sent to the Pavement Design Engineer for review and approval. Upon signing and dating each Typical Section Sheet, the sheets are returned to the designer to be incorporated into the plans.

Design Memorandum “Pavement Type Selection Process” describes the process for selecting the pavement type (e.g., asphalt, concrete). Collectors and local roads typically do not require a formal pavement type selection.

If the pavement design is three or more years old prior to the project being let to contract, submit a “Request for Traffic Data” to the traffic designer. If the three-year period will end near the scheduled letting date of the project, then the pavement design should be reviewed with newly acquired traffic data. If the pavement design is three or more years old, submit a Pavement Design Request Form with a location map to the Pavement Design Engineer and request a review of the existing pavement design. Include the original pavement design with the project’s signed Typical Section Sheets and updated traffic data.

21.1.9 <u>Request for Traffic Data</u>

Traffic data is needed for a pavement design. If a more accurate count of the trucks and other vehicular traffic is desired, check both the top line “Traffic Loading for Pavement Design” and the second line “Classification Count for Pavement Design” on the Request for Traffic Data Form. If only a traffic loading count is necessary without the classification count, then only check the top line on the form. If the second line is not checked, a historical truck count will still be provided. In all cases, the “Future ADT” under “Controls” should be based on the project design year.

Projects that are described in the STIP by route/road number will require the additional classification count. Other projects will be reviewed on a project-by-project basis. Guidelines for these other type projects are:

- project is on an arterial route,
- road/route has an unusually high ADT, and/or
- road/route is in a particularly high growth area.

If it is unclear whether to do a classification count, request a classification count.

If the percent of trucks is all that is desired, then only check the second line “Classification Count for Pavement Design” and provide the location map.

21.1.10 <u>Cross Sections</u>

Cross sections are developed to the extent that construction limits can be determined for the establishment of the new right-of-way limits. Cross sections for the Right of Way Plans should show the location of the existing ground along with the final roadway template. Include the following on the cross sections:

- the finished roadway surface with the appropriate cross slopes (e.g., normal crown, fully or partially superelevated) and the top of subgrade (do not show cross slopes on curb and gutter sections);

- approximate limits of the removal of unsuitable material, as necessary;

- superelevation notes;

- special ditch notes;
- begin/end project;
- begin/end bridge;
- the location of parallel and cross drainage structures on the appropriate cross sections as necessary (e.g., skewed cross drainage may require the development of a skewed section by interpolating between adjacent cross sections);
- locations of pertinent structures, including footings and/or foundations;
- labels for present and new right of way. Review the *SCDOT Geopak Criteria to Place Right of Way on Cross Sections* for guidance on having GeoPAK automate the placement of right-of-way labels. If changes or revisions are made to the right of way, run the procedure again to update the placement of the labels on the cross sections; and
- Wetlands, as requested by the Environmental Services Office.

For more detailed information on cross sections, see Section 22.2.21.

21.1.11 Intersections

In coordination with the traffic designer, intersection details should include all of the elements of the intersection design criteria as described in Chapter 9 “Intersections.” The Intersection Detail Sheets should be in graphical format at a scale commensurate with project complexity. All elements of the intersection should be shown on a single plan sheet so that the entire improvement can be reviewed by observing the overall layout. This may be accomplished by changing the scale of the drawing or making a composite drawing from two or more sheets. The Intersection Detail Sheets should show the roadway intersection angle, number of lanes and pavement width of all legs of the intersection including proposed taper, lengths for turning lanes and proposed radii for curb or edge of pavements. For complex intersections, consider providing a schematic of the design traffic turning volumes on the Plan and Profile Sheets, as appropriate. Sidewalk locations are shown graphically on the Plan Sheets.

21.1.12 Interchanges

Interchange Detail Sheets should show all of the necessary geometry to define and to ascertain right-of-way limits with respect to the geometric ties to other roadways within the interchange area. Include all dimensions identifying pavement widths, pavement transition locations and pavement gore width and locations. These plans should contain a profile for each individual interchange ramp. The Interchange Detail Sheets should:

- provide a graphic geometric layout of all ramps;
- include the horizontal geometry for the mainline, crossroads and ramps;
- show the locations of grade separation structures;

- identify the station and offset ties to the main roadways at locations where they depart and merge with the interchanging main roadways;
- show the edge of mainline traveled way from the beginning or ending of the taper to the location of the gore between mainline and ramp; and
- show completed contours that reflect the grading scheme, sight distance, earthwork, drainage systems, etc.

Due to the complexity of interchanges, each interchange along the proposed project corridor should have a separate Interchange Plan Sheet.

Once a schematic of the proposed interchange improvements is developed, submit it to the traffic designer for the development of a preliminary Signing Plan.

For more detailed information on interchanges, see Chapter 10 “Interchanges.”

21.1.13 Bridge Projects

The structural designer will establish the preliminary type, size and location of project structures as well as loadings, span arrangements and lengths of individual spans. The roadway designer uses the bridge superstructure depths provided by the structural designer to finalize grades in order to provide the proper vertical clearances. Show on the profile the applicable hydraulic data (e.g., design storm, high-water elevation, water velocity) and need for scour protection in addition to the vertical and horizontal clearances. The roadway designer coordinates with the hydraulic designer and structural designer to determine the minimum finished grade elevation.

21.1.14 Drainage

21.1.14.1 Required Information for Hydraulic Studies

The following identifies the roadway information needed by hydraulic designer for various work types:

For Bridge Hydrology Study:

Information needed to begin the bridge hydrology study:

1. alignment (road and bridge);
2. existing profiles (existing finish grade, ground line under bridge, 30-foot left and right profiles, commonly referred to as the triple profile);
3. existing templates;
4. creek traverse (with cross sections); and
5. any other outfall surveys (with cross sections).

With this information, the hydraulic designer will determine and provide the roadway designer with the “Minimum Finished Grade” of the roadway and the “Span and Bridge Length to include the Begin and End Bridge Stations.”

Information needed to continue the bridge hydrology study:

1. The plotted proposed finished grade (with templates including ditches).

<u>For Roadway Hydrology Study</u>:

The roadway designer should provide the hydraulic designer with hard copies of the following plan sheets:

1. Cross Sections, to scale, on half size sheets for the:

    a. mainline,
    b. side roads, and
    c. outfall ditches.

2. Plan Sheets, to scale, on half size sheets showing the:

    a. centerline final grades for mainline,
    b. final grades for side roads,
    c. flow line profiles for all outfall ditches,
    d. limits of construction line, and
    e. all existing survey pipe information.

Reference the *SCDOT Geopak Drainage Manual for Roadway Design* to determine the requirements for the exchange of information between the roadway designer and the hydraulic designer.

Upon receipt of the hydraulic design and sediment and erosion control plans, the roadway designer will set-up a meeting with the hydraulic designer and their consultants, if applicable, to review in detail their plan. This will enhance the transfer of information to the roadway designer in order to improve its inclusion in the roadway plans.

21.1.14.2 Drainage Design

Drainage studies must be sufficiently complete to depict the planned drainage improvements affecting the proposed project (e.g., drainage patterns and features, structures and outfall ditch work).

Show drainage structures (e.g., bridge, pipes, box culvert), ditches/channels, conveyance of storm drainage at side roads and driveways, clean out requirements of transverse ditches/ streams and all sediment and erosion control measures on the plans. Finite data (e.g., lengths of pipe, ultimate pipe elevations) may be omitted from the drawings because these values may change slightly during the final stages of design. The Final Right of Way Plans should also show the limits work and, the proposed methods to accomplish the work (e.g., obtaining permission, acquiring new right of way).

The hydraulic designer is responsible for the design of major crossings and median drainage systems; their design should take into consideration comments received during the Design Field Review. The hydraulic designer also designs cross and parallel storm drainage systems for curb and gutter sections. The roadway designer incorporates the hydraulic designer's drainage design into the plans.

21.1.14.3 Storm Drainage

The roadway designer will notify the hydraulic designer if there are any changes to the top-of-curb elevations, road templates, horizontal or vertical alignments, shape files or typical sections.

Based on this information, the hydraulic designer is responsible for all work related to the design of a drainage system. This includes:

- flow calculations in the system,
- pipe size and material,
- spacing of inlets,
- pipe slopes and invert elevations,
- outfall location and design,
- erosion control and revetment, and
- extra depth box.

The roadway designer will confirm the exact location of inlets to ensure that the inlets are located at low spots and to avoid conflicts with utilities, curb ramps, etc. The roadway designer will perform the drafting and labeling appropriately for all storm drain facilities.

21.1.14.4 Roadside Ditches

The roadway designer determines the dimensions of the roadside ditch based on the criteria presented in Chapters 7 and 14 through 18 of the *SCDOT Roadway Design Manual*. Typically, no analysis is performed to determine hydraulic capacity. However, where necessary, the hydraulic designer will evaluate the ditch and the potential for erosion and, if needed, recommend a ditch size and a permanent protective ditch lining.

21.1.14.5 New Box Culverts

The roadway designer should coordinate with the hydraulic designer and structural designer to determine the minimum cover depth and the size of box culvert to establish vertical alignment. Include the hydraulic data on the plans.

21.1.14.6 Roadway Drainage Criteria

The *SCDOT Requirements for Hydraulic Design Studies* and other documents on the SCDOT's internet site document the Department's hydrologic and hydraulic criteria for the design of roadway drainage appurtenances. The following sections discuss elements of special interest to the roadway designer.

21.1.14.6.1 General

Most highway projects require new drainage facilities and/or the improvement of existing drainage systems. This may be in the form of earth or lined, channels, streams, culverts, closed drainage systems, etc. A specific project may incorporate any or all of these drainage requirements. The roadway designer must be knowledgeable of Department drainage policies and practices affecting roadway design elements.

21.1.14.6.2 Inlet Spacing

Inlets are required at locations needed to collect runoff to meet the design controls specified by the Department's design criteria (e.g., allowable water spread, design year). In addition, there are many locations where inlets may be necessary without regard to contributing drainage area. These locations should be marked on the plans prior to any computations of discharge, water spread, inlet capacity or bypass:

1. Place inlets at low points (e.g., sags) and at intersections as required to intercept the flow.
2. Unless a hydraulic analysis indicates otherwise, space inlets as indicated on the inlet spacing charts provided on the SCDOT's internet site.
3. Place inlets upstream of median breaks, entrance/exit ramp gores, crosswalks and cross slope transitions.
4. Place inlets immediately upstream and downstream of bridges.
5. Re-space inlets following the Design Field Review, as necessary.
6. Other drainage items may be used to supplement catch basins and expand the efficiency and capacity of drainage (e.g., trench drains or extended throats).

21.1.14.6.3 Sideline Pipes

Sideline pipes are identified as longitudinal pipe culverts in roadway ditches at driveways and other locations. Department policy for establishing pipe lengths for standard drives is as follows:

1. Provide a minimum length of 24 feet of pipe at each standard 12-foot drive.
2. Additional pipe (of various sizes) may be shown in the plans as an inclusion item (to be determined during the Design Field Review).

3. If additional (or less) pipe length is required at driveways during construction, the Resident Construction Engineer will make these determinations.

For the use of pipe end structures with respect to roadside safety, reference guidance for “Parallel Drainage Features” found in the AASHTO *Roadside Design Guide*.

21.1.14.6.4 Crossline Pipes

For the use of pipe end structures with respect to roadside safety, review the guidance provided in the AASHTO *Roadside Design Guide*.

21.1.14.6.5 Pipe Alternatives

SCDOT policy is to allow the contractor to select the appropriate pipe material that meets the necessary design criteria. Design Memorandum “Selection of Drainage Pipe for use in South Carolina” provides the allowable pipe types for permanent and temporary installations. For the design purposes, the designer should note the following:

1. Run GeoPAK drainage assuming a smooth wall pipe. Use these results to determine pipe lengths, quantities and pay items.

2. On the Plan and Profile Sheets and the Drainage Detail Sheets, only indicate smooth wall pipes.

3. On the Summary of Estimated Quantities, insert the quantities for only the smooth wall pipe. These quantities are also inserted into the SCDOT Project Programming System (P2S).

4. Include the special provision “Smooth Wall Pipe” with the contract documents.

Do not substitute smooth wall pipe in cases where perforated pipe, pipe underdrain or corrugated aluminum alloy pipe are indicated in the plans.

21.1.14.6.6 Asphalt Gutters

Use Figure 21.1-B to determine the limits of asphalt gutters.

Gutter Design Grade	Distance to Carry Surface Runoff Before Beginning Asphalt Gutter
0 to 0.5%	None Required
>0.5% to 1.00%	1,000 linear feet
>1.00% to 2.00%	500 linear feet
Over 2.00%	250 linear feet

ASPHALT GUTTERS
Figure 21.1-B

21.1.14.6.7 Box Culvert Extensions

A minimum space of 10 feet should be provided between the wing wall ends of box culvert extensions and the present or new right of way. Additional right of way should be provided, as required at each site, to encompass permanent erosion control devices (e.g., energy dissipators, paved liners) placed at the ends of box culvert extensions.

In some cases, the designer may be responsible for designing large junction boxes for box culverts.

21.1.14.6.8 Drainage for Bridge Ends

Good drainage design at the ends of bridges is essential for proper drainage. At bridge ends where the approach roadway does not have curb and gutter and the longitudinal slope of the roadway is less than 1 percent, the typical Department practice is to use an asphalt flume. If the longitudinal slope of the roadway is 1 percent or greater, the designer must determine if the asphalt flume is appropriate. If the asphalt flume is not appropriate, an alternative design must be used to accommodate the runoff.

In addition to an asphalt flume, bridge end drainage may be designed with grate inlets, curb opening inlets or combination inlets. The hydraulic characteristics of the inlets should be considered in selecting the type. Inlets on the bridge should be spaced to minimize runoff entering and exiting the bridge approaches. Collectors at the downslope end of the bridge should be designed to collect all of the flow not intercepted by the bridge deck inlets. If there are no bridge deck inlets, downslope inlets should be provided to intercept all of the bridge drainage. A pipe, paved channel or flume should be used to transport the water down the surface of the embankment.

At bridge ends where the approach roadway has curb and gutter, catch basins should be detailed as close as practical to the approach slabs.

21.1.14.6.9 Trench Drains

Consider providing trench drains where surface flows are suspected to interfere with traffic operations. Runoff from an adjacent property through a driveway toward the roadway can be intercepted by a trench drain installed across the driveway and conveyed into the parallel ditch or into a drainage box. In this case, the pay item “Trench Drain – 8” Interior Dimension (Driveway Application)” may be used.

In curb-and-gutter sections, the typical section provides for runoff to reach the gutter. However, when rehabilitating and widening a section of roadway that was previously a ditch section, but is now being designed as a curb-and-gutter section, grades, vertical curves and superelevation rotation can prohibit conveying the runoff to the desired catch basins and storm sewers. Typically, the minimum desired gutter grade is 0.5 percent; however, 0.3 percent may be used with adequate cross slope. The length of curve can create relatively flat locations on a crest and in a sag vertical curve. Where feasible, catch basin spacing may be reduced to facilitate the efficiency of the drainage system.

Where additional pipe and catch basins are not feasible or the area is not conducive to a catch basin (e.g., at driveways), trench drains may be installed in the gutters to enhance the roadway drainage. Trench drains in gutters will reduce potential ponding in the gutter area caused by inherent, nearly flat grades occurring in pavement transitions and in the low and high points of vertical curves. Typically, the flow line of a trench drain is fixed at 0.6 percent, but will vary according to the grade of the gutter. Trench drains can be placed in an opposing direction to the gutter grade, if the gutter grade does not exceed 0.2 percent in the opposite direction. For example, this would yield a trench drain flow line grade of 0.4 percent in a gutter with an opposing grade of 0.2 percent. This composite grade of the trench drain flow line should not be less than 0.4 percent.

Consider the following guidelines where trench drains are used to supplement drainage in gutters:

- Trench drains may be used where calculated individual longitudinal grades in the gutter are less than or equal to 0.1 percent. Check the actual elevations on the profile to determine the percent grade in vertical curves.
- Provide a drainage box within 96 linear feet to outlet the trench drain.
- Trench drains are designed in 16-foot increments. The maximum length of trench drain in one run is 96 linear feet.
- Include the location and quantity information on the General Construction Note Sheet.

Do not overlap quantities for trench drain and curb and gutter. Where trench drain is extended through a driveway in the gutter, only measure the trench drain where the curb and gutter normally is measured. This is typically in drives where the curb drops to the gutter elevation and does not use a curb radius to follow the edge of the driveway. For a driveway where the curb follows a radius away from the roadway and the trench drain extends into or through the driveway, then measure the trench drain that is not in the curb and gutter and use the pay item “Trench Drain (Driveway).” Deduct the width of the trench drain, including the standard concrete width for the drain, from the area measurement for concrete driveway.

21.1.15 Other Design Elements

Other design elements that should be addressed in the Design Field Review Plans include:

- access management;
- pedestrian/bike accommodations;
- utility impacts;
- traffic management (e.g., detouring, staged construction);
- environmental considerations;
- special design details (e.g., paving under guardrail); and
- water quality.

21.1.16 Cost Estimate

The Program Manager will be responsible for updating the preliminary engineering, right of way, utilities and construction cost estimates based on the Design Field Review Plans.

21.1.17 Preliminary Right of Way Field Review

When Right of Way Plans are approximately 85 percent complete, another field review is recommended. The plans at this stage of development will include hydrology, erosion control, final alignment information, right-of-way lines and control of access. Cross Section Sheets will be complete with superelevation and special ditches. Side road plan sheets should be completed to the extent of the mainline plans. The Right of Way Data Sheet may not be completed.

The purpose of this review is to examine the Preliminary Right of Way Plans with emphasis on roadway drainage and on the impact of the project to the adjacent properties. In addition, this review will identify how new and existing right of way and roadway designs will impact construction.

The Preliminary Right of Way Plans Field Review Title Sheet should be used for all Preliminary Right of Way Plans field reviews.

When the plans are ready for this field review, the Design Group will send electronically created PDF plans to the Engineering Reproduction Services (ERS) to be made available through the Plans Library prior to the field review. The Design Group will notify the review team that the field review plans are available prior to the field review. After the field review, the marked-up plans will be taken by the Design Group to ERS to be scanned making them available through the Plans Library.

21.1.18 Environmental

The Program Manager initiates the environmental process by submitting the Project Planning Report and the Design Field Review Plans to the Environmental Services Office. Guidance for environmental procedures required for roadway projects is provided on SCDOT Environmental Management's internet site and in the *SCDOT Environment Reference Document*. The designer is responsible for ensuring the environmental permits conditions match the final plans.

21.1.19 Minor Structures

The Final Right of Way Plans should show other minor structures (e.g., retaining walls, noise barriers) in the plan and cross section views. Show all information on the plans that depict the intent of the improvements.

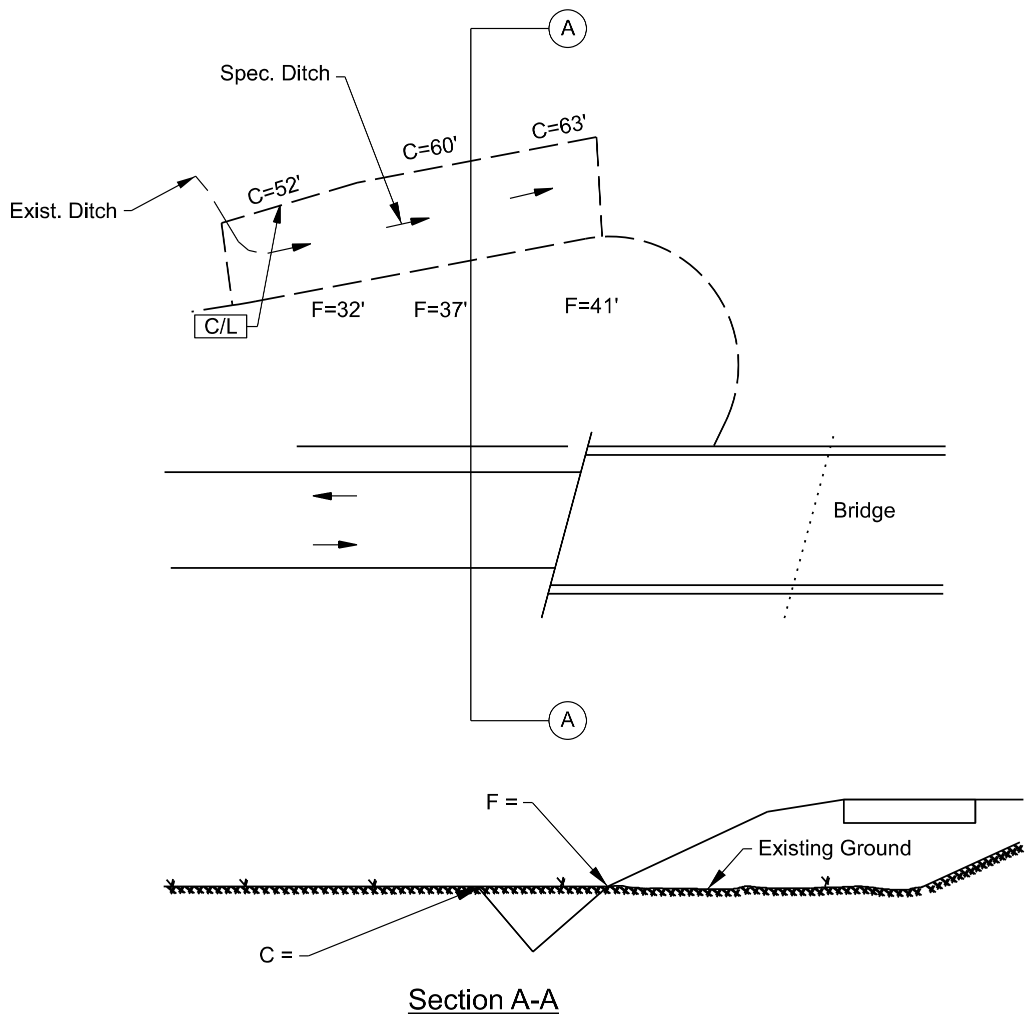
Coordinate with the traffic designer to determine required additional right of way for overhead sign structure footings, signal footings and conventional signing on the plans.

21.1.20 Railroads

The Design Field Review Plans should identify the railroad companies that have facilities in the project area and whether any railroad crossings are to be at-grade or grade-separated structures. All work on railroad right of way must be approved by the Railroad and is coordinated through the Utilities Office.

21.1.21 Construction Limits

The Final Right of Way Plans should show the construction limits in the plan view for each cross section location. Show the construction limits adjacent to interchange ramps, crossroads and parallel drainage courses. In certain cases, it may be desirable to show dual construction limits for clarification purposes at ditch locations or when special ditches are used. Figure 21.1-C provides an example of dual construction limits.



DUAL CONSTRUCTION LIMITS
Figure 21.1-C

21.1.22 Restoration of Construction Slopes in Urban and/or Residential Locations

In areas of concentrated residences, ensure required items of LIKE KIND seeding and/or sodding are included to properly restore established lawns to their preconstruction status. This information should have been obtained during the Design Field Review. Do not address additional considerations or compensation for restoring lawns during the Final Right of Way Plans Phase.

21.1.23 Detours

If detours are deemed necessary for highway constructability, they should be developed during the preparation of the Right of Way Plans. The designer should establish and document the work limits on the drawings. Clearly show the permission to construct, maintain and remove detour on the Right of Way and Construction Plans. Ensure the plans provide adequate quantities to perform the work.

21.1.24 Revision to Final Right of Way Plans

The roadway designer submits copies of the final Right of Way Plans to the Preconstruction Support Operations Center to begin the acquisition of right of way. At this point, the acquisition of right of way begins. The roadway designer continues to complete the plans for construction. Whenever it is necessary to modify the plans, send out copies of the revised sheets so that all current plan holders are aware of the changes, especially to those units that are acquiring right of way or obtaining permissions for the construction.

After the plans have been sent to the Rights of Way Office, the designer is to evaluate the effect of the modifications on the adjacent properties and their functionality (e.g., ingress and egress, appearances and uses). It is the responsibility of the Program Manager to track all changes or revisions to the plans after the initial submittal that will affect the right-of-way acquisition. When right-of-way revisions are necessary during the development of the final plans, notify the Rights of Way Office immediately.

It is important that every revised sheet contain a note in the revision box (or upper right-hand corner of sheet if no revision box) that includes the date, initials of the person making the change and a brief description and location of the change, so that anyone who receives a copy of the revised sheet can easily see what has changed from the original sheet.

Ensure that any revision affecting other features of the project (e.g., utilities, wetlands, traffic control) is distributed to the respective Section. This process continues until the plans are completed for construction.

Do not submit changes to the plans that do not affect the right-of-way process to the Rights of Way Office.

21.1.25 Construction Office Plan Review

When the plans are approximately 75 percent complete and the quantities are essentially complete, submit the plans, including the Summary of Estimated Quantities Sheet, to the Resident

Construction Engineer assigned to the project. If a Resident Construction Engineer is not assigned, then the District Construction Engineer will review the plans.

The purpose of this review is to examine the plans for completeness with emphasis on the estimated quantities and/or pay items necessary to construct the project. A field review may be held, but is not required. If a field review is requested or determined by the designer to be necessary, then the designer will set-up and lead the field review.

A special Summary of Estimated Quantities Sheet is available for the Construction Office Plan Review on the SCDOT's internet site. This sheet allows the reviewer to make recommended changes to quantities and pay items directly on the sheet. Only use this Sheet for this review (i.e., do not use this Sheet in the final plans).

21.1.26 Borrow Pit Locations and Monitoring

Highway and bridge projects often require borrow materials (soils) to be brought to the project site by the contractor for the construction of roadway embankments. It is the contractor's responsibility to provide the borrow materials that are required for the project. These borrow materials may be acquired from appropriately permitted, commercially operated borrow pits or from borrow pits located and established by the contractor with appropriate permits. In order to avoid or minimize the impacts of borrow pits on wetlands, the Program Manager needs to ensure the borrow material has been appropriately screened for wetland materials. Design Memorandum 30 "Borrow Pit Location and Monitoring" outlines the procedures for when and where screening will be required on a project.

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21.2 FINAL DESIGN

21.2.1 Construction Plans

21.2.1.1 Purpose

The Construction Plans are a collection of all the design plans from all the applicable units (e.g., Road, Traffic, Structures, Hydraulic, Geotechnical). After they are reviewed, approved and signed by the applicable individuals, they are forwarded to the Preconstruction Support Operations Center for letting and ultimately to the contractor for construction.

The final Construction Plans incorporate all of the amendments or revisions evolving from the final roadway plan review. Additionally, the designer will complete all of the quantity tables and identify any special provisions necessary for the construction of the project. The designer should coordinate with the Rights of Way Office to review all right-of-way special provisions to ensure that all commitments made during right-of-way negotiations are incorporated into the plans and/or Special Provisions, as appropriate.

21.2.1.2 Title Sheet

The Construction Plans Title Sheet is developed to reflect the proposed work represented by the plan drawings. This sheet may differ from the Right of Way Plan Title Sheet (e.g., sheet numbering, sheet indexing, project mileage).

Use the signed Title Sheet from the Right of Way Plans to develop the Title Sheet for the Construction Plans. Occasionally, Right of Way Plans are developed for the entire project while the Construction Plans are developed for the same project in phases.

21.2.1.3 Revisions

If a revision is necessary during the development of the Construction Plans that may affect the designs or work of another unit (e.g., Traffic, Rights of Way Office), notify the applicable unit immediately and furnish them with the revised sheets.

21.2.1.4 Architect-Engineer Plan Errors and Omissions

Design Memorandum "Architect-Engineer Plan Errors and Omissions" provides the Department policy to process and document plan errors and omissions. It also provides guidelines for recovering additional costs to a project or damages to the Department that may be attributable to plan errors or omissions.

21.2.2 Plans, Specifications and Estimates

21.2.2.1 Specifications

All plans include, by reference, the latest edition of the *SCDOT Standard Specifications for Highway Construction*. The following definitions apply:

1. <u>Standard Specifications</u>. Specifications is the general term comprising all the directions, provisions and requirements contained in the *SCDOT Standard Specifications for Highway Construction*, together with such as may be added or adopted as supplemental specifications, or as special provisions and all documents of any description, including notes on plans, pertaining to the method and manner of performing the work or to the quantities and qualities of materials to be furnished under the contract.

2. <u>Supplemental Specifications</u>. Supplemental Specifications are specifications adopted subsequent to the publication of the *SCDOT Standard Specifications* and constitute a part thereof and of the contract. Supplemental Specifications prevail over *SCDOT Standard Specifications* when in conflict therewith.

3. <u>Supplemental Technical Specifications</u>. Supplemental Technical Specifications are specifications adopted for construction items that change frequently due to new technologies and constitute a part thereof the contract. Supplemental Technical Specifications prevail over *SCDOT Standard Specifications* and Supplemental Specifications when in conflict therewith.

4. <u>Special Provisions</u>. Special provisions are provisions inserted in the proposal form and contract revising or supplementing the *SCDOT Standard Specifications* to cover conditions peculiar to the individual project. Special provisions take precedence over *SCDOT Standard Specifications* and Supplemental Specifications.

   Special provisions for all road and bridge projects are prepared by the Engineer of Record.

Inquiries regarding *SCDOT Standard Specifications*, Supplemental Specifications or special provisions as related to plan preparation may be directed to the Specifications and Estimates Office.

21.2.2.2 Engineer's Estimate

The Lettings Preparation Engineer is responsible for preparing the Engineer's estimate. The Engineer's Estimate provides the Department with a basis for evaluating the bids for highway construction and allows the Department to determine if the low bid price is fair and reasonable for the work involved. This estimate, plus the data used to generate the estimate, is considered confidential and is not for general distribution.

21.2.2.3 Final Proposal

The Letting Preparation Engineer is responsible for preparing the Bid Proposal. The Bid Proposal document will include:

- Notice to Contractors,
- Instructions to bidders (Federal Projects),
- Project Special Provisions,
- Supplemental Specifications,
- required Contract Provisions for Federal-aid construction projects (Federal Projects),

- standard Federal Equal Employment construction contract specifications (Federal Projects),
- wage regulations (Federal Projects),
- bid bond,
- proposal form, and
- DBE Committal Sheet (Federal Projects).

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21.3 CONSTRUCTION PHASE

21.3.1 Construction Bids

The Contracts Administration Office is responsible for advertising projects, construction bid openings, reviewing bids and awarding projects. Once the contract has been accepted and signed, the contractor can begin construction on the project. At this project stage, the designer may be requested to clarify the construction plans, offer guidance, review shop drawings, etc.

21.3.2 Construction Plans Storage

The Construction Plans are filed with the Plans Storage Office and are to be kept in permanent storage. They are only revised to reflect major changes that occurred during construction.

Plans Library is SCDOT's archived access via the internet to as-let and as-built road construction plans of highways on the State Highway System. Plans are placed into an electronic archive after projects are awarded to contract, which is typically within one month of the highway letting. Access to Plans Online is available to surveyors, consultants and other entities based on a yearly subscription. The library includes nearly every set of roadway plans since 1908. For security reasons, bridge plans can only be requested by submitting a request form to the Plans Storage Office.

21.3.3 Final As-Built Plans

During construction of the project, the Resident Construction Engineer maintains a separate set of drawings on which all revisions to the plans on the project are recorded. These plans, when checked and completed, are transmitted to the Plans Storage Office where they are kept for permanent record. The *SCDOT Manual of Instructions for the Preparation of As-Built Plans* provides guidance on the preparation of as-built plans.

21.3.4 Plan Revisions and Construction Changes

21.3.4.1 Revised Plan Sheets

Use the following procedures to revise the plan sheets after they are submitted to the Operations Center:

1. Changes Prior to Letting. Changes to plan sheets that are made available to the bidders prior to the highway letting will be reviewed by the responsible designer, who will verify that all changes have been made in the CADD files. Revised plan sheets will be provided to the Letting Preparation Engineer and then to the Engineering Reproduction Manager who will incorporate the revised sheets into the plans after the letting date.

2. Changes After Letting. Revised plan sheets that are not available to the bidders prior to the letting will not be added to the Bid Plans at any time. These revised sheets will be submitted to the Preconstruction Support Operations Center and will be added to the As-Built Plans when construction is completed. This procedure is for all revisions made after the letting or when the revisions cannot be made available to the prospective bidders prior

to letting. The designer labels any additional or revised plan sheets provided with ½-inch bold letters under the box in the upper, right-hand corner, CONSTRUCTION CHANGE-SHEET PROVIDED AFTER LETTING. If no space is available under the box, then any location near the box is acceptable. These sheets will not be added to the Bid Plans, but will be placed face up in the back of the stored plans for reference. Revised and added plan sheets during the construction phase will be incorporated into the final As-Built Plans after construction is completed.

21.3.4.2 Working Plans

Working Plans requested by District personnel include all revised sheets in the proper order with the old sheets removed. The Engineering Reproduction Manager will maintain a complete copy of the Working Plans and will provide copies to Department employees when specifically requested. The Engineering Reproduction Manager will mark the plans with WORKING PLANS and with the date of printing. The plans are clearly marked to avoid confusing these plans with the Bid Plans or the final As-Built Plans.

21.3.4.3 Revisions to Working Plans

After the letting, the bid plans are filed in the Plans Storage Office. After the award of the project, it may be necessary to make changes or revisions to the bid plans. Once a change is made, the bid plans are now called Working Plans. Changes to the working plans are made by roadway designer. Revised sheets are provided to Operations by roadway designer. If the change will affect right of way or a property owner, also submit the revised sheets to the Rights of Way Office.

It is important that every revised sheet contain a note in the revision box (or upper right-hand corner of sheet if no revision box is provided), which includes the date, initials of the person making the change and a brief description and location of the change, so that anyone who receives a copy of the revised sheet can easily see what has changed from the original sheet. Also, place a note stating “CONSTRUCTION CHANGE – SHEET PROVIDED AFTER LETTING” on each revised sheet.

The revised sheets are scanned by Engineering Reproduction Services. It is important to maintain an up-to-date scanned copy of the working plans. Requests for copies of plans are submitted to the Plans Storage Office by numerous personnel from within and outside the agency. It is vital that the copies they receive are current and match all other copies of the working plans. As revisions are made, the Engineering Reproduction Services Manager adds a note to the title sheet of the working plans (“Working Plans as of DD/MM/YY”) to record the revisions contained in the working copy.

The original hard copies of the revised sheets are then returned by Operations to the roadway designer to be inserted into the hard copy of the working plans, which are then returned to Plans Storage Office.

This process continues until the project is complete. The working plans then become the final set of “As Designed” plans. These plans incorporate the original bid plans with all of the revisions made and approved by the designer. This set is filed in the Plans Storage Office.

The set that is stored in the Plan Storage Office is not an as-built set. The As-built Plans are completed by SCDOT District personnel or the contractor and submitted to the Final Plans Section of the SCDOT Director of Construction Office where they are filed and stored. It is important to note that the designer cannot be responsible for all changes or revisions to plans. During the construction of the project, the Resident Construction Engineer assumes responsibility for changes that arise to meet field conditions. It is the responsibility of the Resident Construction Engineer to track and record these changes on the As-built Plans. Changes by the field offices are not sent to Operations for copying, distribution, scanning, etc.

21.3.4.4 Title Sheet and Estimated Quantity Revisions

Any plan sheet may be revised except for the Title Sheet. If the Title Sheet needs to be changed, contact Roadway Design Support.

Show revised quantities on a revised Summary of Estimated Quantities Sheet. The original quantities must not be changed. Add a note to the Quantity Sheet describing which plan sheets are affected by the change or additional work.

The Engineering Reproduction Services Manager will archive all plan sheets, original and revised, on the appropriate Department’s server. The CADD Support Manager will archive all design files, including the revisions.

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Chapter 22

PLAN SHEETS PREPARATION

SOUTH CAROLINA ROADWAY DESIGN MANUAL

March 2017

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Chapter 22
PLAN SHEETS PREPARATION

22.1 GENERAL

The accompanying chapters in the *SCDOT Roadway Design Manual* provide the designer with uniform criteria and procedures for the geometric design of a highway facility. These designs must be incorporated into the plans so that they can be clearly understood by contractors, material suppliers and Department personnel assigned to supervise and inspect the construction of the project. To ensure a consistent interpretation of the plans, individual sheets should have a standard format and content, and the sequence of plan assembly should generally be the same. Therefore, guidelines have been established for the uniform preparation of plans including, drafting guidelines, recommended plan sequence, plan sheet content and sample plan sheets.

A certain degree of discretion must be exercised in the development of plans. Simple projects may include additional details on the basic plan and profile drawings and projects that are more complex may require separate special drawings in the final product.

Any major deviations from the instructions contained in this chapter should be coordinated with the designer's supervisor and Preconstruction Support.

22.1.1 <u>Drafting Guidelines</u>

22.1.1.1 Computerized Drafting

Computers perform the tedious tasks of drawing cross sections, plotting mass diagrams, calculating grading quantities, etc. They also allow the designer the freedom to develop and evaluate alternatives. However, they require the designer to be well versed in complex software.

The Department's CADD standard is MicroStation and GEOPAK. All CADD work must be developed using MicroStation and GEOPAK software. For more information on CADD symbols, line styles and CADD cells used by the Department, see the SCDOT CADD Design internet site.

The following sections provide information on the design software, drafting criteria and the CADD file management used by the Department to prepare its plans. See the Department's website for further information.

22.1.1.2 Internet

SCDOT provides various road design resources online on the SCDOT website. All Instructional Bulletins, *SCDOT Standard Specifications*, *SCDOT Standard Drawings*, Pay Items and CADD workspace files can be viewed at this website.

22.1.1.3 Electronic Media Deliverables for Consultant Projects

The *SCDOT Road Design Reference Materials for Consultant Prepared Plans* is included in the Scope of Services on all consultant contracts that include roadway design plan deliverables. See the Department's design memorandum for more information.

22.1.2 Abbreviations

There are recognized abbreviations commonly used in the drafting of highway plans. Figure 22.1-A contains an alphabetical listing of the abbreviations that are to be used in preparation of highway plans for the Department. Acronyms that commonly appear on the drawings are also included in this alphabetical listing.

DESIGNATION	ABBREVIATION
-A-	
Abandoned	Abd.
Abutment	Abut.
Acre	Ac.
Addition	Add.
Adjust	Adj.
Aggregate	Agg.
Ahead	Ah.
Alternate or Alternative	Alt.
Angle	Ang.
Apartment	Apt.
Approach	Appr.
Approximate	Approx.
Asphalt	Asph.
Auxiliary	Aux.
Avenue	Ave.
Average	Avg.
Average Annual Daily Traffic	AADT
Average Daily Traffic	ADT
Azimuth	Az.
-B-	
Back	Bk.
Backsight	BS
Baseline	B/L
Basement	Bsm't
Bearing	Brg.
Benchmark	B.M.
Bituminous	Bit.
Block	Blk.
Book	Bk.
Borrow	Bor.
Boulevard	Blvd.
Brick	Br.
Bridge Construction Access	BCA
Building	Bldg.
Business	Bus.
-C-	
Cable Television	CATV
Capacity	Cap.
Cast Iron	C.I.
Catch Basin	CB

DESIGNATION	ABBREVIATION
Cemetery	Cem.
Center	Ctr.
Centerline	C/L or
Chord	Chd.
Church	Ch.
Circle	Cir.
Circumference	Circum.
Clean Out	C.O.
Company	Co.
Concrete	Conc.
Connection	Conn.
Construction	Const.
Controlled Access	CA or C/A
Corner	Cor.
Corrugated Aluminum Alloy	CAA
Corrugated Metal Pipe	CMP
Corrugated Polyethylene	Corr. P.E.
County	Co.
Creek	Ck.
Cross Section	X-Sect.
Cubic Feet (Foot)	CF
Cubic Feet Per Second	CFS
Cubic Yard	CY
Culvert	Culv.
Curb and Gutter	C & G
-D-	
Decimal	Dec.
Degree	Deg. or (°)
Delta	Δ
Department	Dept.
Design	Dgn.
Design Hourly Volume	DHV
Diagonal	Diag.
Diameter	Diam. or Φ
Dimension	Dim.
Direction	Dir.
Distance	Dist.
Drawing	Dwg.
Drive	Dr.
Ductile Iron	D.I.
Dwelling	Dwlg.

PLAN ABBREVIATIONS
Figure 22.1-A

DESIGNATION	ABBREVIATION
-E-	
Each	Ea.
Easement	Easm't
East	E.
Edge of Pavement	EOP/EP
Edge of Traveled Way	ETW
Elbow	Ell.
Elevation	Elev.
Engineer	Engr.
End of Information	EOI
Equality or Equation	Eq.
Estimate	Est.
Excavation	Exc.
Existing	Exist.
Expansion	Exp.
External	E
-F-	
Face of Curb	FOC
Face Curb to Face Curb	FC-FC
Feet or Foot	Ft. or (′)
Feet Per Second	fps
Fiber Optic Line	FOL
Figure	Fig.
Fire Hydrant	FH
Finished Grade	FG
Flat Bottom	FB
Flow Line	FL
Foresight	FS
Frame	Fr.
-G-	
Gallon	Gal.
Gallons Per Minute	GPM
Garage	Gar.
Gauge or Gage	Ga.
Grade	Gr.
Guard Rail	GR
-H-	
Headwall	Hdwl.
High Point	H.P.
High Water	HW
Highway	Hwy.
Horizontal	Horiz.

DESIGNATION	ABBREVIATION
-I-	
Inch	In. or ″
Including	Inc.
Inside Diameter	ID
Invert	Inv.
Iron Pin	IP
Iron Pin New	IN
Iron Pin Old	IO
Irrigation	Irrig.
-J-	
Joint	Jt.
Junction Box	JB
-L-	
Lateral	Lat.
Left	Lt.
Length of Curve	L
Length of Tangent	T
Light Pole	LP
Linear Feet	LF
Low Water	L.W.
Lump Sum	LS
-M-	
Manhole	M.H.
Masonry	Mas.
Material	Mat'l
Maximum	Max.
Mean Low Water	MLW
Mean High Water	MHW
Mean Sea Level	MSL
Miles Per Hour	MPH
Minimum	Min.
Minute	Min. or (′)
Miscellaneous	Misc.
Monument	Mon.
-N-	
National Pollutant Discharge Elimination System	NPDES
Necessary	Nec.
North	N.
Not in Contact	N.I.C.
Not to Scale	NTS
Number	No. or #

DESIGNATION	ABBREVIATION
-O-	
On Center	O.C.
Ordinary High Water	O.H.W.
Outside Diameter	OD
-P-	
Page	Pg.
Palmetto Utility Protection Service	PUPS
Parcel	Par.
Pavement	Pvm’t or Pv’t
PCC	
Percent	%
Perforated	Perf.
Permanent	Perm.
Present	Pres.
Point of Compound Curve	PCC
Point of Curve	POC
Horizontal	P.C.
Vertical	V.P.C.
Point of Intersection	
Horizontal	P.I.
Vertical	V.P.I.
Point of Reverse Curves	PRC
Point on Tangent	POT
-R-	
Radius	R
Reinforced Concrete	RC
Remove	Rem.
Required	Req’d
Residence	Res.
Retain	Ret.
Revise (Revision)	Rev.
Right	Rt.
Right of Way	R/W
Road	Rd.
Roadway	Rdwy.
Route	Rte.
-S-	
Sanitary Sewer	S.S.
Second	Sec. or ″
sheet	Sh.
Sidewalk	S/W or Sdwlk.
Signal	Sig.

DESIGNATION	ABBREVIATION
Smooth Wall Pipe	s.w. pipe
South	S.
Square	Sq.`
Square Foot	SF
Square Yard	SY
Standard	Std.
Station	Sta.
Storm Sewer	St. S.
Story	Sty.
Street	St.
Structure	Str.
Subsurface Utility Engineering	SUE
Superelevation	S.E.
Superelevation Rate	e
Surface	Surf.
Survey	Surv.
Symbol	Sym.
System	Sys.
-T-	
Tangent	Tan.
Telephone	Tel.
Telephone Pedestal	Tel. Ped.
Telephone Pole	TP
Television	TV
Temporary	Temp.
Terminal	Term.
Thousand	M.
Thousand Square Yards	MSY
Top of Bank	T.O.B.
Topography	Topo.
Typical	Typ.
Typical Section	T.S.
-U-	
Unclassified	Uncl.
Underdrain	U-drain
Underground Storage Tank	UST
Unknown	Unk.
-V-	
Valve	V.
Vehicle	Veh.
Vertical	Vert.
Vitrified Clay Pipe	VC Pipe

DESIGNATION	**ABBREVIATION**
Volume	Vol.
-W-	
Water Line	W.L.
West	W.
White	Wh.
With	w/
Without	w/o
-Y-	
Yard	Yd.
Year	Yr.
Yellow	Yel.

22.1.3 Sheet Organization

Figure 22.1-B provides plan sheet numbers that are assigned to certain drawings and a listing of additional drawings to be incorporated into each number set.

Most highway and bridge project plans are similar as far as their basic content. As the project is designed, drawings are organized systematically and much of the information required to produce the plans is similar. Some complex projects require additional detail, or more in-depth study or design, which results in additional drawings or data not common to most projects. In this case, the recommended procedure for sheet organization and numbering in Figure 22.1-B may require alteration. Alterations should be discussed with and approved by the designer's supervisor during the early planning stages.

If road profiles are shown on a separate sheet, the Profile Sheet should follow the corresponding plan sheet and be numbered as an "A" sheet. For example, if Sheet 6 is the Plan Sheet, the next sheet, the Road Profile Sheet, would be Sheet 6A.

Only the Title Sheet contains the total plan sheet count. Show this number in the box in the upper right-hand corner of the Title Sheet. As the plan development process proceeds through Right of Way and Construction Plans, the number of plan sheets will change. Revise the number in box accordingly.

Upon completion of the Construction Plans, the number shown in this box will reflect the sum of the total sheet count as contained in the Index of Sheets.

22.1.4 Contract/Project ID

A Contract Identification (Contract ID) number is generated by P2S when a project is added to a letting. Multiple projects may be combined into a single contract. Before the plans are uploaded into the Plans Library System, the Engineering Reproduction Services Office will place a watermark stamp on the Title Sheet of each set of plans depicting the Contract ID.

Project Identification (Project ID) numbers on the plans must match P2S Project ID numbers. This should be monitored closely because an incorrect identification number can cause problems with the use of the Plans Library after the project is awarded. If there is any concern that the Project ID number may be wrong, verify it with the Program Manager before submitting plans to the Roadway Design Support for a quality assurance review.

When referencing previous projects that do not have a Project ID number, use the original file number when preparing verification notes for existing right of way and/or miscellaneous references notes to historic projects.

22.1.5 Sealing Engineering Documents

Design Memorandum "Standards for Sealing Engineering Documents" provides the Department policy for signing and sealing construction documents. It provides guidelines for which engineer should seal the documents, what documents are to be sealed and what plan sheets are required to have seals.

Sheet Number	Description
1, 1A, etc.	Title Sheet
IL1, IL2	Index/Layout of Sheets
2, 2A, etc.	Summary of Estimated Quantities, Demolition and Moving Items, Reset/New Fences, etc.
3, 3A, etc.	Typical Sections and Miscellaneous Details (not covered by *SCDOT Roadway Standard Drawings*)
4, 4A, etc.	Right of Way Data Sheet, Property Strip Map, Property Closure Sheets
5	General Construction Notes Sheet
5A, etc.	Reference Data Sheets, Station Offset Sheets
6, 7, 8, etc.	Plan and Profile Sheets (e.g., mainline, side roads, ramps)
6A, 7A, 8A, etc. *****	Profile Sheets (e.g., TOC, barrier)
D1, D2, D3, etc. *****	Drainage Plan Sheets
12, 13, 14, etc.	Details of Plan Sheets, Geometric and Grading Plan for Intersections and Interchanges, Intersection Detail Sheet
TC1, TC2, etc.	Traffic Control and Construction Phasing, Detour Plan and Profile Sheets
E1, E2, etc.	Electrical and Lighting Plans
L1, L2, etc.	Landscaping Plans
PM1, PM2, etc.	Pavement Marking Plans
SN1, SN2, etc.	Signing Plans
TS1, TS2, etc.	Traffic Signal Plans
S1, S2, etc.	Roadway Structure Plans (e.g., retaining walls, box culverts)
G1, G2, etc.	Geotechnical Details/Ground Improvement Methods
EC1, EC2, etc.	Erosion Control Data Sheets
U1, U2, etc.	Utility/Utility Relocation Sheets (e.g., SUE, utility information, Utility Relocation Sheet)
X1, X2, etc.	Cross Sections
XP1, XP2, etc.	Crossline Pipe on Special Cross Sections
BR1, BR2, etc.	Bridge Plans (only if included with roadway plans)
UC1, UC2, etc.	Utility Construction Sheets

***** *Note that these sheets should only be used where the information cannot be combined with the Plan Sheet*.

SHEET NUMBERING SYSTEM
Figure 22.1-B

22.2 CONSTRUCTION PLAN SHEET CONTENT

The criteria discussed in this section for data presentation and content on drawings are intended to produce uniformity in the development of Construction Plans. The result is a set of plans that is easy to organize, review, check and quantify. Following established procedures both reduces time and cost and, ultimately, facilitates the construction of projects.

The plans should be prepared as simply as practical. Avoid the use of duplicated data and unnecessary cross references. This section and the sample plan sheets on the SCDOT Roadway Design internet site provide guidance on what should be included within each sheet of a set of Construction Plans. The SCDOT CADD Design internet site provides information on text sizes, font types, symbols, cell libraries and drafting levels that should be used to develop a set of Construction Plans.

22.2.1 Plan Cover

The purpose of a Plan Cover is for easy referencing to the project's general information and for filing accessibility. The Plan Cover should display the county, Project ID No., Route No. or Road No., and the termini. This information should match the information on the Title Sheet. See the Department's intranet for guidance on in-house projects and Design Memorandum "Electronic Media Deliverables for Consultant Projects" for consultant projects.

22.2.2 Title Sheet (Sheets 1, 1A, etc.)

The Department has master Title Sheets that are to be used on all roadway projects — one for in-house designed projects and one for consultant-designed projects. The following information is contained on the base drawing:

1. Heading. The heading is centered directly under the SCDOT logo. It should include the county, Project ID number, road or route number, road name (if available) and termini.

2. Project ID Numbers. Show Project ID numbers on the Title Sheet in their designated locations.

3. Road Names. Use road names with road numbers, if available.

4. Termini. Ensure the termini describe the project accurately and completely.

5. Location Map. The location map is placed directly under the heading and should contain enough information (e.g., city, town, landmarks) so the project can be easily located. Accurately show the proposed project with a wide, heavy line. Provide an arrow showing the direction of the survey along the proposed project. Position the map so that North is straight up on the page, if reasonable. Always include a North arrow. Provide a note with arrows pointing to the beginning and ending of the project indicating the Project ID number, road or route number, beginning and ending station and sheet numbers of plan and profile on the location map.

6. Exceptions. Include a note, where applicable, for exceptions to the project showing Sta.___to Sta.___, length of exception and what the exception is (e.g., road, railroad,

bridge in place) with an arrow pointing to the exception on the location map. Locate the notes to best use the available space on the Title Sheet.

7. New Bridges. Provide a note for new bridges to be constructed that identifies the bridge length, type (e.g., precast, prestressed, R.C.), Sta.___to Sta.___ and Project ID number of bridge, if it is being let under a separate Project ID number.

8. Map. Show the scale of the map below the words “Layout” directly under the map. Also, provide the map name under the map (e.g., RICHLAND CO., CITY OF COLUMBIA).

9. Mileage Box. Below the map, accurately complete the mileage box. When binding more than one road with separate Project ID numbers together, provide separate mileage for each road. On all projects, identify the main line mileage, side road mileage, ramps, etc., and show a separate column with total mileage. Gross length of project should be Total Length including exceptions and bridges. Subtract out the length of exceptions to get the Net Length of project and subtract out bridges to get the Net Length of roadway.

   A detour route is the use of adjacent roads to redirect traffic from a road that is closed due to construction activities. For detour routes that require permanent improvements (e.g., resurfacing, widening, pavement markings), calculate the mileage of the detour route, show this mileage on the Title Sheet and add it to the total project mileage. If the work requires staged construction, lane shifts for traffic, or temporary alignments within the project limits (e.g., bridge or culvert replacement projects), do not include the detour mileage in the project mileage.

10. Equalities. List any equalities in stationing under the mileage box. For example: Sta. 10+15.5 Back = Sta. 13+25.5 Ahead (-310′) or Sta. 10+15.5 Back = Sta. 7+75.5 Ahead (+240′). If there are no equalities, show the word “None.”

11. Traffic Data. All projects must have present Traffic Data shown in the blanks provided. For projects requiring a pavement design, also include the Traffic Data for the Design Year and the percent of trucks as shown in the pavement design.

12. Index of Sheets. If room is available, show the Index of Sheets in the upper left corner. See 22.1-B for sheet organization and sheet numbering. For large projects, a separate page for the Index of Sheets may be used; see Section 22.2.3.

13. Permit Information Box. For NPDES purposes, space has been provided to show the disturbed area, the permitted area, the project’s latitude and longitude for both the beginning and ending of the project and to identify who provided the hydraulic and NPDES design.

14. Railroads. Complete the railroad involvement box by circling YES if a railroad is involved or NO if none are involved.

15. Right of Way Plans. Right of Way Plans should have an approval signature located on the lower right portion of the Title Sheet. For right-of-way acquisition, a professional engineering seal will not accompany this signature because these plans are considered preliminary, unless specified otherwise by the consultant contract. The right-of-way approval signature will be accompanied by the printed name of that person, their title and

the date. The Regional Production Engineer or designee is the appropriate signature. This is also true for consultant prepared plans.

16. PE Signature/Seal. The final construction plans will be signed, dated and sealed by the engineer of record using the space allocated above the signature block on the lower right portion of the Title Sheet.

17. Specification. The note shown at the bottom center of the Title Sheet specifies the year edition of the *SCDOT Standard Specifications* and *SCDOT Standard Drawings* used on the project. The year may be changed as appropriate.

18. Hydraulic Design. Reference the design criteria used for hydraulic design on each set of plans in a note added to the Title Sheet. Place the note close to the Permit Information Box.

19. Environment Permit Information Box. The roadway designer should coordinate with the Program Manager to determine the environmental permits that are appropriate for the project. Place an "X" on the blank line to reflect the appropriate permits required for the project. Check all navigable water permits that are applicable to the project or select "N/A" if a navigable water permit is not applicable.

20. Call 811 Box. Ensure the CALL 811 box is shown on the Title Sheet.

21. Design Criteria. Identify whether the project was designed using new construction/ reconstruction criteria or 3R criteria.

Basic information may change for different projects and the designer needs to be aware of revisions in order to keep the project Title Sheet up to date.

22.2.3 Index of Sheets

The Index of Sheets is generally placed in the upper left corner of the Title Sheet. The numbering system should include all sheets identified with a letter suffix. Sheet descriptions should be brief and informative. Methodology of sheet numbering is addressed in Figure 22.1-B.

The designer may elect to remove the Index of Sheets from Title Sheet and place it on a sheet called the Index/Layout Sheet. Locate this sheet immediately after the Title Sheet and number it as sheet IL1. Additional sheets may be used and should be numbered sequentially as IL2, IL3, etc. This sheet will contain both the Index of Sheets and a layout/overview of the project with the sheet numbers shown.

Show the total sheets in a series of numbers in a third column to the right of the sheet descriptions on the last line of the series. If a series of sheets preceding the plan and profile sheets are omitted, still include the sheet number, but add the word "Omitted."

22.2.4 Summary of Estimated Quantities, Removal and Disposal Items, Reset/New Fence Sheets, etc. (Sheets 2, 2A, etc.)

The Department has developed standard summary sheets for the development of all roadway projects. These are the Summary of Estimated Quantities, Removal and Disposal Items, and Reset/New Fence Sheets.

Projects with more than one road and the same Project ID number may list the quantities separately, but must have a total quantity column. Projects that have more than one Project ID number bound together must have a total quantity column for each Project ID number. Also, projects that are located in more than one county must have separate quantities for each county.

22.2.4.1 Roadway Summary of Estimated Quantities Sheet

The Roadway Summary of Estimated Quantities Sheet is discussed in the following sections. For guidance on developing quantities, see Chapter 20 "Quantities." Every item needed to construct the project should be shown on the summary of estimated quantities sheet unless provided elsewhere (e.g., landscaping plants, moving items).

22.2.4.1.1 Project Programming System

The Project Programming System (P2S) is designed to provide all agency users with a quick and reliable source for gathering, maintaining and reporting all pertinent project information from beginning to end. P2S integrates with other agency applications (e.g., ITMS, Primavera, CBES, WebTrns•port, SiteManager) to make finding project and contract data more organized.

This system provides a process for assigning numbers to each bid (pay) item and is under the direction of the Preconstruction Support Engineer. The system requires all bid (pay) items to be coded by a seven-digit number (e.g., #7204100 – Concrete Sidewalk 4″ Uniform). The first three digits of the numbering system refer to the particular section of the SCDOT *Standard Specifications*. The last four digits of the numbering system are unique to the particular bid (pay) item.

The designer should coordinate with the Lettings Preparation Engineer to obtain a new bid item number if one has not been previously assigned.

The designer should quantify those items applicable to the project and identify all items by the assigned seven-digit number. Upon completion of the quantity listing, a final Summary of Estimated Quantities Sheet(s) is computer generated and incorporated into the plans.

22.2.4.1.2 Pay Items Not in Plans

Certain pay items in a contract may require a contractor to perform work, but specific instructions or details are not shown in the plans. Generally, these items are requested by the District during the Design Field Review or are procedural on each project. The designer should specify the pay items in Figure 22.2-A as inclusions on the General Construction Notes Sheet and include the corresponding work descriptions for each item.

Pay Item	Pay Unit	Note
MOBILIZATION	LS	Per Contract Documents
MOBILIZATION	EA	Per Contract Documents
MOBILIZATION – SUBCONTRACTOR	LS	Per Contract Documents
BRIDGE CONSTRUCTION ACCESS	LS	Per Contract Documents
BONDS AND INSURANCE	LS	Per Contract Documents
CONST. STAKES, LINES & GRADES	EA	Per Contract Documents
CONSTRUCTION STAKES, LINES & GRADES (BRIDGE ONLY)	EA	Per Contract Documents
CPM PROGRESS SCHEDULE	LS	Per Contract Documents
BORROW PIT SET-UP	LS	Per Contract Documents

PAY ITEMS NOT IN PLANS
Figure 22.2-A

22.2.4.1.3 Revisions

When revising plans that have already been let and awarded and there are revised quantities, provide a revised Summary of Estimated Quantities Sheet along with the revised plan sheet(s). Figure 22.2-B provides an example of the note to be placed below the original quantities. Do not change the original quantities. The note should include the date of the revision and the sheet numbers of the revised sheets. The changes should be listed in the order as shown in Figure 22.2-B, as applicable. An addition symbol (+) and subtraction symbol (−) depict the quantities, as appropriate, on the Summary of Estimated Quantities Sheet.

Future revisions are placed beneath the previous revision. Additional Summary of Estimated Quantities Sheets may be added, if necessary. See Chapter 21 “Procedures for Highway Plans Preparation” for procedures related to plan changes.

22.2.4.1.4 Structures

Quantities for non-bridge structures are shown on a detail sheet. Final quantities itemized on the detail sheet are included on the Summary of Estimated Quantities Sheet.

Item No	Pay Item	Quantity	Pay Unit
1031000	MOBILIZATION	NEC.	LS
1050800	CONST. STAKES, LINES & GRADES	1	EA
1071000	TRAFFIC CONTROL	NEC.	LS
1090200	AS-BUILT CONSTRUCTION PLANS	NEC.	LS
2012000	CLEAR & GRUB WITHIN RDWY.	NEC.	LS
2013050	CLEARING & GRUBBING DITCHES	0.500	ACRE
2031000	UNCLASSIFIED EXCAVATION	4165.000	CY
2033000	BORROW EXCAVATION	7797.000	CY
2034000	MUCK EXCAVATION	3717.000	CY
3050108	GRADED AGGR. BASE COURSE – 8″ UNIF	10215.000	SY
3069900	MAINTENANCE STONE	50.000	TON
3103000	H/M ASPH. AGG. BASE CR. – TYPE 2	2746.000	TON
4010005	PRIME COAT	2759.000	GAL
4011004	LIQUID ASPHALT BINDER – PG64-22	232.000	TON
4013990	MILL EXIST ASPH. PVMT. – VARIABLE	2820.000	SY
4023000	H/M ASPH. CON. BINDER CR. – TYPE 2	991.000	TON
	Revised Pay Items – 07/24/15		
	Effected Sheets 6,7,8		
	Pay Items Revised	**Adjustments to Quantities**	
2034000	MUCK EXCAVATION	-500.000	CY
3069900	MAINTENANCE STONE	+25.000	TON
	Pay Items Deleted		
4013990	MILL EXIST ASPH. PVMT. – VARIABLE	-2820.000	SY
	Pay Items Added		
4031100	H/M ASPH. CONC. SURF. CR. – TYPE 1	+1013.000	TON

REVISED SUMMARY OF ESTIMATED QUANTITIES
Figure 22.2-B

22.2.4.1.5 Pavement Marking, Lighting, Signing and Traffic Signals

Because the traffic designer is responsible for the design and development of Pavement Markings, Roadway and Bridge Lighting, Roadway and Bridge Signing and Traffic Signal Plans, traffic control element quantities are developed by the traffic designer and supplied to the roadway designer for incorporation on the final Summary of Estimated Quantities Sheet. If not provided by the traffic designer, include a quantity for permanent construction signs. If pavement markings are not provided by the traffic designer, include pavement markings and raised pavement markers. See *SCDOT Standard Drawings* for guidance.

22.2.4.2 Removal and Disposal Items, Moving Items, Reset/New Fence Sheets

22.2.4.2.1 Submissions

The Rights of Way Office provides a list of all items to be moved, removed and disposed of or reset during the construction of a project to the roadway designer. This information is based on negotiations with the landowners.

Moving items and removal and disposal items are submitted electronically. In addition, fencing and underground storage tanks quantities are submitted electronically, except for decorative fencing (e.g., picket, wrought iron, brick). Decorative fencing should be included as moving item.

It is especially important that data concerning septic tanks, drain fields, underground storage tanks or any other similar item be accurately obtained in order to ensure these items are backfilled during construction.

All projects requiring items of work involving demolition, moving and new or reset fence are incorporated into these sub-summary sheets.

22.2.4.2.2 Table Content

The column designated “ITEM NO.” is used to designate a certain item. Each item number may contain more than one particular moving item. One item number should contain all moving items for each tract or property owner. This is also true for the Removal and Disposal Items.

If combining more than one project item together in a set of plans, begin each item with Moving Item No. 1. If there is more than one road in an item, continue numbering the items concurrently. This also applies to Removal and Disposal Items.

Always leave a space between each Moving Item and Disposal Item, even if it requires more than one page.

Complete the columns designated “LOCATION” by showing the location of the item by the station and the offset distance and whether to the left or right of the centerline.

In the columns entitled “DESCRIPTION” and “WORK TO BE DONE,” designers must ensure that complete and detailed information is provided. This is necessary so that the construction contractor fully understands what is expected and the Department and contractor do not have a dispute over the work expectations. If the work description becomes too lengthy, it may be necessary to prepare a special provision that is incorporated into the plans.

The column designated “UNIT” will always contain the initials L.S. (Lump Sum) and the column designated “OWNER” should contain the property owner name.

Include reset fence item, shown in a lump sum amount, on the Moving Item Sheet and on the Summary of Estimated Quantities Sheet. Show new fencing for a specific location on the Plan Sheets and on the Summary of Estimated Quantities Sheet.

Only show moving items/demolition items on the Moving/Demolition Item Sheet. Do not show moving/demolition items on Plan Sheets.

The roadway designer should note that all quantities summarized on these specific sheets are transferred directly to the project proposal by the Letting Preparation Engineer and are not shown on the Summary of Estimated Quantity Sheet. Do not incorporate the schedules of prices on the Summary of Estimated Quantities.

22.2.4.2.3 Underground Storage Tanks

Underground Storage Tanks (UST) are set up as Removal and Disposal Items. They may be combined with other Removal and Disposal Items on the same tract, but must contain their own location (i.e., station number and offset Lt. or Rt.), accurate description (e.g., 10,000 GALLON KEROSENE TANK) and work to be done (e.g., REMOVE AND DISPOSE OF TANK ACCORDING TO DHEC REGULATIONS). If the locations of UST are known, plot them on the Plan Sheets.

The right-of-way agent should provide any items regarding UST. Include the following UST items on the General Construction Notes as well as the Summary of Estimated Quantities Sheet:

- Removal and Disposal of Tank Contents,
- Removal and Disposal of Low Level Contaminated Soil, and/or
- Removal and Disposal of High Level Contaminated Soil.

22.2.5 Typical Section Sheets and Miscellaneous Details (Sheets 3, 3A, etc.)

All Construction Plans must contain Typical Sections that fully describe the intended work to be performed on a project. They are normally developed as full Typical Sections. It is recommended that a single Typical Section Sheet contain no more than two mainline sections. Show side roads and incidental sections on separate sheets. However, more than two side roads or incidental typical sections may be shown on a single sheet.

22.2.5.1 Scale

For clarification of each Typical Section, provide a horizontal scale that uses one-half to two-thirds of the available sheet width. If a section tends to lose vertical definition, the vertical scale should be exaggerated for clarification purposes. If not drawn to scale, label as “NTS.”

22.2.5.2 Dimensions

Most Typical Sections have an element that predominates throughout (e.g., 12-foot lane width). However, if there are conditions where this width may vary, place a note indicating the variable width after the standard dimension (e.g., SHOWN 12 FEET (VARIES 12 FEET TO 24 FEET)). Show all dimensions for travel lanes, curb and gutter, valley gutters, earth and paved shoulders, sidewalks, standard ditch bottoms, base and paving materials, etc. Identify construction centerlines and finish grade points on each section. Show all dimensions in feet and decimals of a foot (e.g., 12.5 feet versus 12½ feet).

22.2.5.3 Identification of Materials and Application Rates

Clearly identify all materials and their depth, which comprise the pavement structure, on each Typical Section.

When identifying the pavement structure on the Typical Section Sheet, include the specified type of mix that is noted in the pavement design. This information needs to be included on the Typical Section Sheet before submitting it to the Pavement Design Engineer for approval.

If the mix is changed during the development of a project, ensure that the Typical Section Sheets are also revised to reflect the change. All revised Typical Section Sheets that originally needed the Pavement Design Engineer's approval must be resubmitted to the Pavement Design Engineer for approval.

22.2.5.4 Station Limits

Typical Sections are developed for the entire project. This is generally accomplished by delineating all station limits, to which the section applies, below each Typical Section. Transition areas (e.g., transitioning from a four-lane divided section to a five-lane section) are indicated under the appropriate Typical Section by station limits followed by brief descriptions of the variable dimensions. Verify these station limits prior to Construction Plans completion.

22.2.5.5 Clarification Notes

The designer is advised to make full use of the additional notes on the Typical Section Sheets to help clarify the intent of each section as it relates to the planned improvements.

22.2.5.6 Partial Sections and Blow-ups

The use of both partial sections and area blow-ups are encouraged to help clarify the design features of the improvements. They may be shown on the Typical Section Sheet to which they apply, or grouped together on an additional Incidental Section Sheet.

22.2.5.7 Curb Ramps

On all projects with sidewalks, provide the identification of each curb ramp on the individual plan sheets. However, include a note on each Typical Section Sheet indicating that curb ramps are to be constructed in accordance with the *SCDOT Standard Drawings*.

22.2.5.8 Design Speed

Include the design speed(s) on the Typical Section Sheet.

22.2.5.9 Functional Classification

Use the appropriate CADD cell and place the functional classification/roadway group designation directly left of the design speed information block on the bottom right corner of the Typical Section Sheet.

22.2.5.10 Special Details

Special details may be required on Typical Sections for roadway projects where an item of work is not specifically designated in the *SCDOT Standard Specifications* or *SCDOT Standard Drawings*. Special details may be in the form of brick paver details, new devices, new technology or new ideas.

22.2.5.11 Superelevation Layouts

The roadway designer may use a superelevation diagram sketch or a superelevation table to assist in the development of the superelevation for a project.

A separate typical section may be shown for superelevation.

22.2.5.12 Miscellaneous Details (Special Drawings)

A special drawing is a drawing that is not typically included on all projects, but is tailored to a particular situation on the project. Examples of special drawings that are stored in the CADD files on the SCDOT intranet site include:

- Milling Details;
- Pavement Concrete Joint (Main Line, Aux Line and Ramp);
- Lighting Standard for Single Slope Concrete Median Barrier;
- Stilling Basin (Riprap Placement at Box Culvert);
- Sediment Basin;
- Special Survey Codes Legend;
- Sound Barrier Wall;
- Retaining Walls;
- Box Culverts;

- Large Drainage Boxes;
- Standard for Replacing Pavement on Backfill Over Pipe in Existing Roadways; and
- Guardrail Application at Bridge with Sidewalk.

Some other special drawings for specific situations that need detailing are riprap placement at stilling wells and wire enclosed rock (Gabion Basket), etc. Special drawings should be included in the plans and may have special provisions written for them.

Special drawings are provided as a baseline only to aid designers with commonly used items. These special details must be completed with appropriate design information before they are included in the plans. It is the responsibility of the designer to ensure proper information is given on these drawings and that they are appropriately used in the plans. Special drawings should include the bid items necessary for construction, where applicable, on the drawing.

22.2.6 Right of Way Data Sheets, Property Strip Map, Property Closure Sheets (Sheets 4, 4A, etc.)

All projects requiring the inclusion of special sheets to document and summarize the right-of-way elements are to be incorporated into the Sheet No. 4 series of the plans.

22.2.6.1 Right of Way Data Sheet

The Right of Way Data Sheet is used to summarize all agreed upon transactions between property owners and the Department. It contains standard headings that, when properly completed, will provide a complete status of affected properties.

The designer should make full use of the comments column to the extent the Right of Way Data Sheet depicts all work anticipated or provides additional data beneficial to the property owner and the right-of-way agent.

Sufficient space is available on the Right of Way Data Sheet to include clarification notes.

Items to be shown on the Right of Way Data Sheet include:

- tract number;
- name, as it appears on instrument of record;
- tax map reference;
- total area in acres or square feet as referenced on the instrument of record;
- obtained area in acres or square feet;
- remainder of area in acres and/or square feet if less than 0.25 acre;
- date of acquisition provided by the Rights of Way Office;
- type of instrument provided by the Rights of Way Office; and
- permissions.

22.2.6.1.1 Present Right of Way

Prior to the Design Field Review, the designer is required to verify the present right of way. The designer should identify the present right of way on the plans with a description note containing the following:

1. Right-of-Way Width. The note should indicate the right-of-way width as measured from the existing centerline.

2. Source. The note should identify the source used for verification. Designate the right of way "Present R/W" if verified from a previous set of plans or deeds. Use the designation "Dedicated R/W" if determined by plats. For example, "Dedicated R/W Plat Book 123 Page 56" indicates that the plat is recorded in the county Register Mesne Conveyance (RMC) or Clerk of Courts Office under Plat Book 123 at Page 56. The source will be the File Number where the right of way was originally obtained.

   If previous project plans are not available, seek assistance from the Rights of Way Office to determine the present right of way and show the documentation information of that source on the plan sheet.

3. No Source. In the event that no right of way can be determined, contact the Director of Rights of Way to make a determination.

On the Plan Sheet, place the Present R/W note as follows:

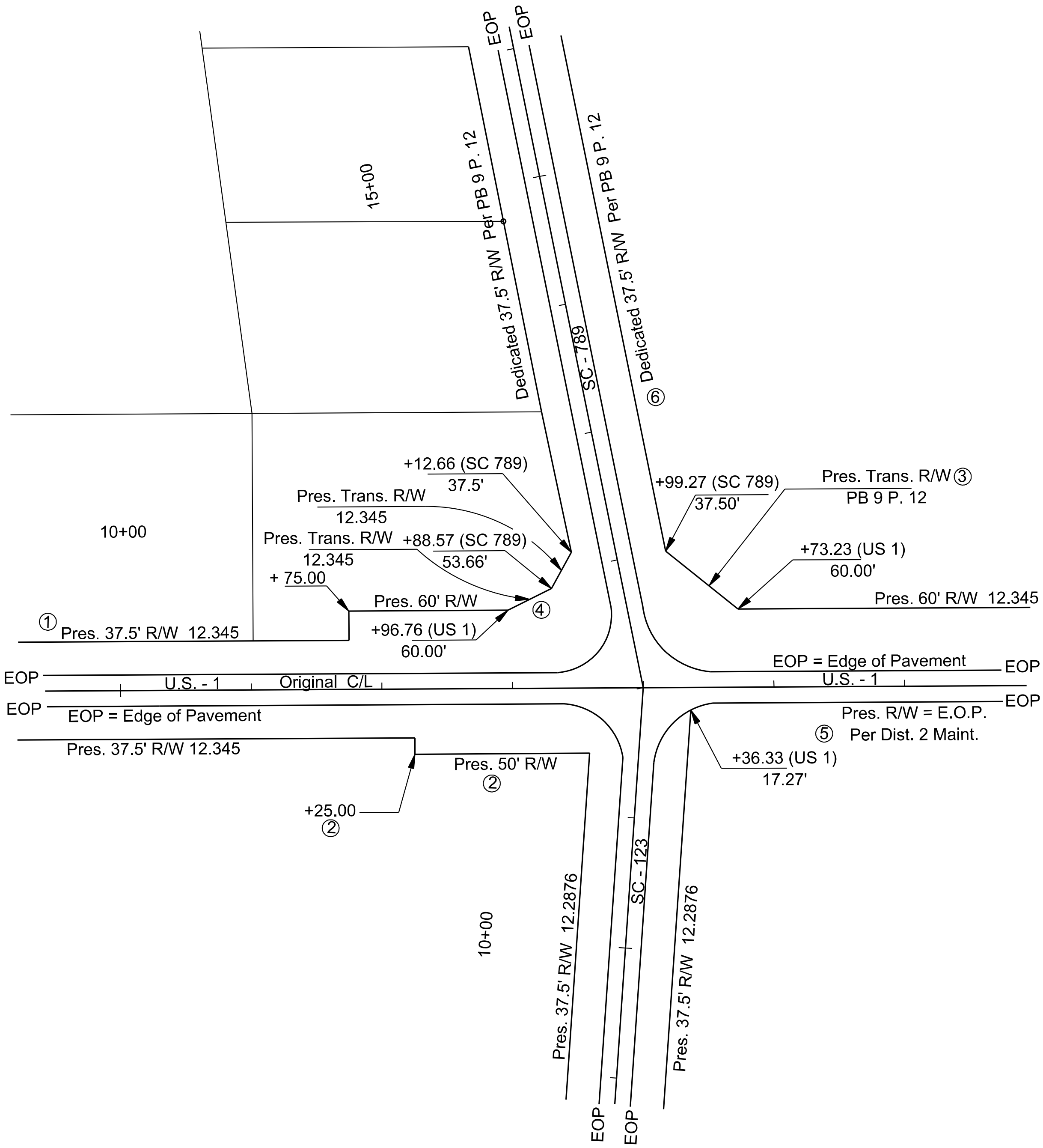
1. Place the note on the mainline adjacent to the right-of-way line at the beginning and ending of each sheet on the left and right of the centerline; see ① in Figure 22.2-C. This placement verifies that the right of way shown from that station forward is from that source.

2. If the present right-of-way width changes under the same source, place a label specifying the changed right-of-way width adjacent to the changed right-of-way line. Include a note specifying the station. See ② in Figure 22.2-C.

3. For standard triangular areas, label the station offset and associated route number at each change in the direction of the right-of-way line. Label the triangle line as "Pres. Trans. R/W" and include the source; see ③ in Figure 22.2-C. For non-standard triangular areas with multiple vertices, include additional labels for the intermediate vertices and show the station, associated route number and the offset; see ④ in Figure 22.2-C. It will no longer be necessary to use "Δ" area labels.

4. Place a new note verifying Present R/W along the right of way each time the source changes; see ⑤ in Figure 22.2-C.

5. Place the R/W note for side roads adjacent to the side road right of way on the left and right of the side road centerline; see ⑥in Figure 22.2-C.



PRESENT R/W NOTATION
Figure 22.2-C

22.2.6.1.2 New Right of Way

The normal identification of a new right-of-way line will be via the width from centerline notation. Hence, the line noting new right of way on the left side of a roadway centered about a proposed 100-foot right-of-way width will be shown as “NEW 50′ R/W.” See Chapter 12 “Right of Way” for examples.

22.2.6.1.3 Breaks in Right of Way

Changes between varying widths of right of way may be achieved through the 90-degree method or the transition method. The 90-degree method entails a double offset at a single station along the alignment (40 feet left of Station 71+00 and 30 feet left of Station 71+00). The transition method entails a single offset at a pair of unlike stations along the alignment (40 feet left of Station 71+00 and 30 feet left of Station 81+00).

22.2.6.1.4 Identification of Property Features and Improvements

Those elements that clearly define a property may include, but are not limited to, property lines, subdivision boundaries, buildings, trees, shrubs, flower beds, water wells, septic tanks, leech fields, retaining walls, drives, sidewalks, outbuildings, garages, swimming pools, etc. All or any one of these items may be critical to, or instrumental in, the establishment of new right-of-way boundaries. All are important items as they relate to property appraisals. Show and clearly reference their characteristics for each of these items on the Right of Way Plans. For example, the limits of a drain field are shown; width and material type indicated for sidewalks and drives; width, height and material type indicated for retaining walls; height and material type indicated for fences; number of stories, type and usage of buildings (e.g., two-story frame residential or one story brick commercial), etc. Items such as underground storage tanks (USTs), landfills, dumpsites or other features that may have environmental implications should also be identified by size and location.

Determine the limits of topographic identification during the request for survey stage of plan development. A general guide to the limits of topography should be in the range of 50 feet to 100 feet beyond the anticipated new right-of-way boundaries. This must be reviewed and analyzed on a project-by-project basis.

22.2.6.2 Property Strip Map

Develop a Property Strip Map for inclusion in the plans. If individual property closures are shown on separate sheets, incorporate them into the Sheet No. 4 series of the plans. Items to be shown on the Property Strip Map include:

- tract number;
- all present and new right of way and property lines labeled;
- existing man-made and natural features (e.g., road, alignments, outfalls, streams, lakes, cities, towns, landmarks);
- property inserts may be included with a scale;
- assigned parcel numbers;

- any breaks in the Control Access or Limited Access Line;
- horizontal alignment; and
- scale.

22.2.6.3 Property Revisions

Any plan revisions made after Right of Way Plans are submitted must be detailed by placing a revision note on the appropriate Plan Sheet. If the revision affects a particular tract, this should be noted. All plan revisions, regardless of their impact on the right of way, are to be routed to the Rights of Way Office in the normal manner to ensure that the right-of-way agent has the correct information to provide to the affected landowners.

22.2.7 General Construction Sheets (Sheets 5, 5A, etc.)

The designer must ensure that typical general construction notes that are commonly used on roadway projects are applicable and accurate for the present project.

22.2.7.1 General Construction Notes

General construction notes are notes to the Resident Construction Engineer and contractor that denote quantities or work not otherwise shown in detail on the plans or *SCDOT Standard Drawings*. The construction note is usually shown on an individual sheet. It communicates instructions and quantities not detailed on the plans. For quantities listed in the inclusions and shown on the General Construction Note Sheet, provide a brief description of how each item is to be used. The following are examples of construction notes:

1. The Deputy Secretary for Engineering must specifically authorize changes involving increased cost of project or changes in alignment. The District Engineering Administrator is permitted under the direction of the Deputy Secretary for Engineering to authorize minor alterations not in conflict with the standard practices of the Department. Forward information on any proposed changes in alignment to the Columbia Office as soon as practical.

2. The following quantities are not shown in detail on the plans, but are included in the Summary of Estimated Quantities and may be adjusted during construction as directed by the Engineer.

3. Pipe lengths that are shown on the plans are actual lengths calculated along the pipe slope from center of box to center of box. Field adjustments of the actual pipe length may be necessary.

To avoid errors and/or misunderstandings in construction plans, describe all items (inclusions) listed on the General Construction Note Sheet exactly as addressed in the Pay Item list. Do not show the bid item number on the General Construction Note Sheet.

Some items listed are self-explanatory; however, others should be itemized in more detail as shown in the example below:

Hot Mix Asphalt Intermediate Course Type A	150 Tons for Build-up 50 Tons for Detours
Maintenance Stone	50 Tons for Drives 50 Tons for Roadway 20 Tons for Pavement Patching

On the sheet, provide a box showing the Program Manager and Roadway Group Manager's names and telephone numbers.

22.2.7.2 Miscellaneous Notes

There are many miscellaneous notes placed throughout the plans (e.g., beginning and ending of curb and gutter, sidewalks, control of access, fences, retaining walls, etc.). The following provides guidance on the use of miscellaneous notes:

1. Bridge Excavation. Occasionally, excavation becomes necessary to construct bridges. Crosshatch the area on the profile and include a note with an estimate of the necessary excavation. See Figure 22.2-F.

2. Cross Reference Notes. Cross references are useful to direct someone to a supplemental sheet or to a continuation of a survey that runs off the Plan Sheet (e.g., Candy St. continued on page 7).

3. Utility Notes. The first Plan Sheet should contain all necessary information relating to all utilities on the project. Place utility notes in the upper left corner of the first plan sheet (e.g., all power poles and lines owned by SCE&G Co.); see Section 22.2.8.4.

4. Typical Section Notes. Only provide notes showing the beginning and ending stations on Typical Section Sheet (e.g., "Use this section on S.C. Route 502 from Sta. 83+47 to Sta. 105+19.").

5. Mucking Notes. Clearly state mucking information on the profile and show the quantities in the inclusions if not calculated from the cross sections. When mucking is shown on the final cross sections, depict the removal line as shown in the *SCDOT Standard Drawings*.

6. Driveway Notes. When the pavement structure for driveways is different from the structure shown on the typical section, include a note stating the driveway structure on the General Construction Note Sheet.

22.2.7.3 Reference Data Sheets

Examples of information shown on Reference Data Sheets include:

- reference points for PC, PI, PT and POTs;
- control points;
- benchmark locations and elevations;
- horizontal curve data with superelevation design criteria;
- vertical curve data for curbs and/or sidewalks;

- station offset and surveyed features;
- survey datum and factor information;
- coordinates (northing, easting and elevation) for survey control points;
- optional top of curb profiles; and
- optional Station/Offset text.

See individual curves on Reference Data Sheet or Plan Sheets for superelevation rate and design speed, as applicable.

22.2.7.4 Drainage Data Sheet

Where the Plan and Profile Sheets are complex, it may be desirable to provide a separate drainage sheet to improve the Plan Sheet's clarity. Number the drainage structure on the Plan Sheet and then describe the drainage structure on the Drainage Data Sheet. This may include the:

- pipe size,
- box size,
- length,
- end type,
- material type,
- end elevations, and
- other special details.

See Section 22.2.8.11 for additional guidance on the drainage information to be included.

22.2.8 Plan and Profile Sheets (Sheets 6, 7, 8, etc.)

The Plan and Profile Sheets are the key elements of a project and must provide project clarity, correctness, orderliness and completeness. A proper drawing scale commensurate with the project's complexity and topography is necessary.

22.2.8.1 Topographic Features

Each Plan and Profile Sheet should contain complete and accurate topographic information. The information is obtained from surveys, field reviews and the Design Field Review process. Topographic information should be concise and to the point so that the existing conditions are well represented and understandable (e.g., when making reference to a residence, the Plans should indicate one story frame residence). The designer should ensure that items (e.g., churches, schools, businesses, service stations with underground tanks, playgrounds, parks, historic monuments, hospitals) are properly identified on the plans. Clearly identify environmentally sensitive areas, in which construction and construction staging activities are to be avoided, using stations and offsets. Identify all highways, streets, drives and parking areas. Include the highway and/or street and route designation. Indicate the existing pavement type for each. Clearly show the existing property and right of way documentation on the plans. For projects where the topographic information is over 5 years old, a decision should be made between the Program Manager, Design Manager and the Surveys Office on whether to secure current data.

The designer should include station/offset text on the plan and profile sheets. The text is created from the survey data and provides station/offset to various topographic features within the project corridor. The designer has the option to include text as part of the reference data sheets instead of including on the plan and profiles sheets.

22.2.8.2 Right of Way and Property Features

For guidance on right of way and property features, see Section 22.2.6.

22.2.8.3 Control of Access

Clearly denote on the Plan and Profile Sheets all routes having fully-controlled access or limited access. Any proposed breaks in the controlled access line (other than access points offered at interchanges and sometimes at intersections) should clearly be identified on the plans in the following manner:

BEGIN C/A STA. 10+50 END C/A STA. 11+40

22.2.8.4 Existing Utilities

Substantial utility data is available from mapping and field surveys. Additional data that may be required can be obtained from the Utility Office including data relative to planned utility improvements and relocations.

Show all existing utilities on the Plan Sheets. Show the names of the various utilities on the first Plan Sheet and the Utility Information Sheet. Dithering is the use of the gray scale in lieu of black to show lines and information on the plan sheet. When separate Drainage Sheets (D sheets) are provided in the plans, the existing utility information will be dithered on the Plan Sheets. On the Drainage Sheets, the existing utility information will not be dithered.

See the *Subsurface Utility Engineering CADD Development Manual* for additional guidance concerning utility plan sheet development.

22.2.8.5 Horizontal and Vertical Alignments

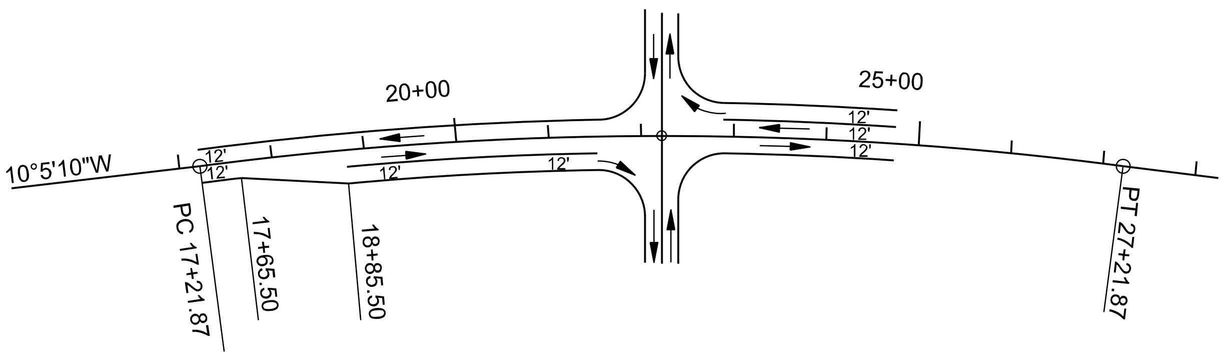
The geometric design of a roadway is the combination of horizontal and vertical alignments. The horizontal alignment is set by curves and tangents and the vertical alignment defines the elevations of the horizontal alignment.

22.2.8.5.1 Horizontal Alignment

All distances are calculated and expressed to the hundredth of a foot. Show all points of curvature (e.g., PC, PT, PRC, PCC) on the centerline or baseline of the horizontal alignment. Tangent

Tangent centerlines should show the alignment bearings expressed by quadrant degrees, minutes and seconds (in the direction of stationing). Station registration is indicated via a tick mark on the centerlines. Stationing is labeled at a minimum of every 500 feet for 50 scale drawings and every 100 feet (i.e., every station) for 20 scale drawings. For the method of station presentation, see the *SCDOT Standard Drawings*.

Figure 22.2-D provides an example of labeling the horizontal geometrics. Show centerline reference points on the Reference Data Sheet and clearly indicate which reference points apply to the appropriate roadway centerlines.



PI STA. 22+24.75
Δ = 15°00'00" RT
D = 1°30'00"
R = 3,819.72'
T = 502.88'
L = 1000.00'
E = 32.96'
DS = 60 mph
$e_{(max)}$ = 8.0%
SE = 4.2%
PC - LG% = 0.5%
PT - LG% = 0.5%

HORIZONTAL GEOMETRICS
Figure 22.2-D

Where it is necessary to relocate the roadway centerline of a project, show the new points for the relocated PC, PI, PT and POTs on the Reference Data Sheet. Identify these new points as relocated points. Each point is identified by xyz coordinates on the Reference Sheet and shown on the Plan Sheet as “RELOCATED PC,” with stationing. This procedure is in lieu of providing station offsets for the relocated points. Use the new coordinate description on all future work. Place a note on the Plan Sheet to bring attention to the relocation. Relocations are also shown with different line styles. Show centerline reference points on the Reference Data Sheet and clearly indicate which reference points apply to the appropriate roadway centerlines.

22.2.8.5.2 Vertical Alignment

All elevations are calculated to a hundredth of a foot. Show the following on the vertical alignment (profile):

1. <u>Tangent Sections</u>. Express all straight line slopes (grades) on the vertical alignment as a positive or negative percentage (i.e., 1-foot rise or fall per 100 feet of horizontal distance).

2. <u>Vertical Curves</u>. Include the following vertical curve data on the profile:

   - the length of curve expressed in feet,
   - the curve beginning (VPC) expressed by stationing with VPC elevation,
   - the curve ending (VPT) expressed by stationing with VPT elevation, and
   - the intersection (VPI) of forward and back tangent grade lines expressed by stationing with VPI elevation.

3. <u>Elevations</u>. Show profile grade elevations to two decimal places for points on the profile at all VPCs and VPTs and at 50-foot intervals throughout vertical curves. In urban areas, it may be necessary to show profile elevations at more frequent intervals. Also, show finished profile grades at all intersection points and at all tie points used in conjunction with cross slope elevations to establish the beginning of ramps, loops or flyover profile elevations.

4. <u>K-Values</u>. Show the design K-value for the vertical curve on the profile.

5. <u>Bridges</u>. Show the structure thickness, vertical clearance and elevation of overpasses.

6. <u>Special Ditches</u>. Show profile with grade elevations at 50-foot intervals.

22.2.8.6 Travel Lane Lines

For the Plan Sheet, the ETW or edge of pavement (EOP) line should be paralleled with a face of curb (FOC) line. Do not show the back of curb line. Where there is a grass buffer strip, show the front edge of the sidewalk. In all cases, show the back of the sidewalk.

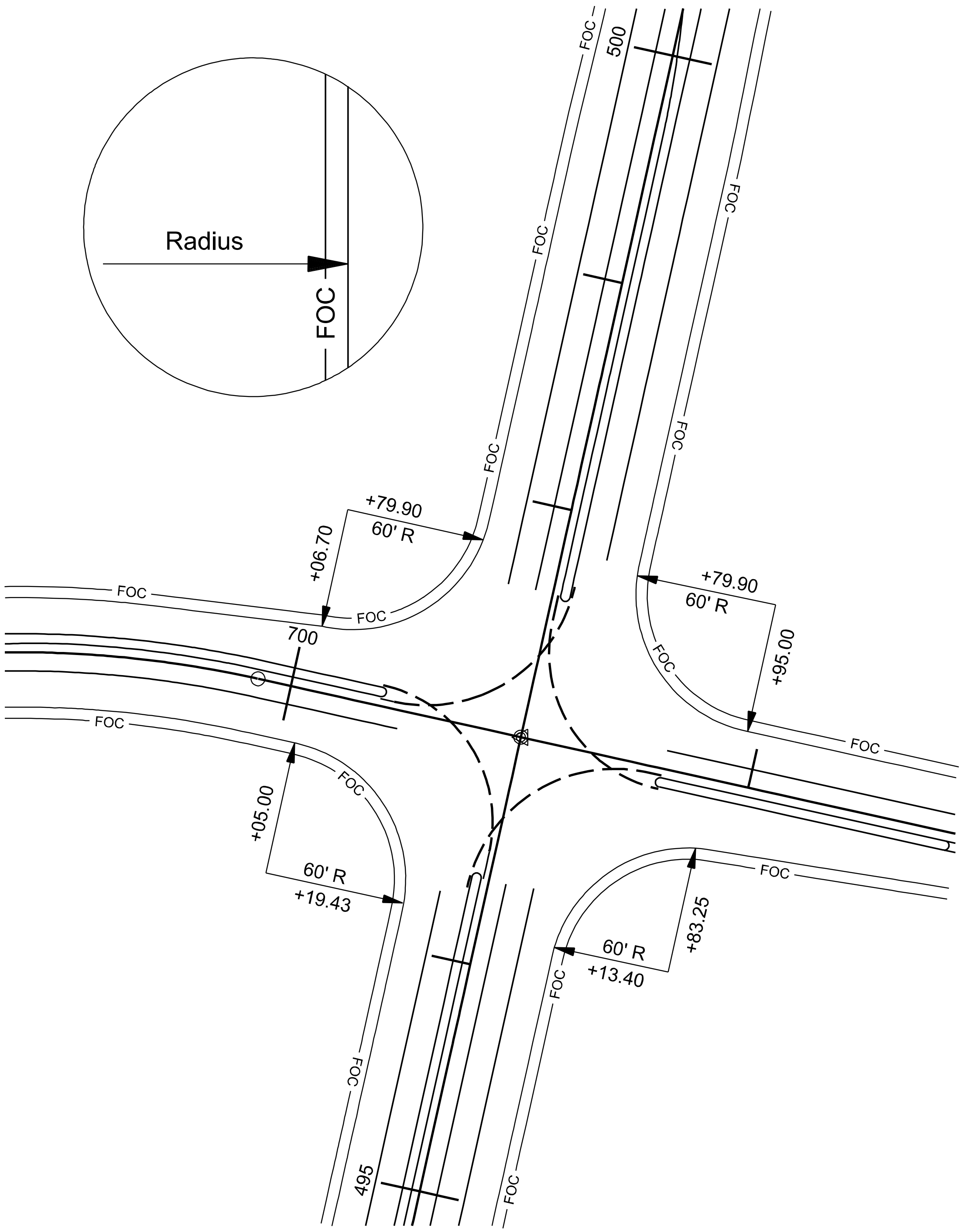
On projects where a valley gutter lip is proposed or where a portion of the shoulder is to be paved only show the travel lane(s) on the plans. Do not indicate the edge of paved shoulder or lip except where a variation from the typical section is necessary. The Typical Section should show the travel lane and dimension.

Draw the radius dimensions to the ETW line on the plans as shown Figure 22.2-E. For clarification, a cell has been created to demonstrate the radius measurement. Place the information on each plan sheet, where applicable, to emphasize the measurement location.

For additional information, see the guidance provided on the SCDOT CADD Design internet site.

22.2.8.7 Proposed Roadway Features

Dimension all the proposed roadway features and relay to the users the exact intent of the proposed roadway improvements. This includes edges of traveled way, face of curb, turning radii, tapers, nosings, medians (flush or raised), paved shoulders (greater than 4 feet in width), median barriers, bridges, sidewalks, etc. Show all widths, offsets and stationing, where appropriate.



DRAFTING CURVE RADII
Figure 22.2-E

22.2.8.8 Sidewalks and Paths

For sidewalks, label widths, radii, transition tapers, offsets, pedestrian ramps and detectable warnings. For paths, also include the horizontal and vertical alignments information.

22.2.8.9 Construction Limits

Indicate construction limits on the plans via a dashed line. Include the distance from the centerline or baseline and the cut/fill notation on the drawings at each location where a cross section has been developed.

22.2.8.10 National Pollutant Discharge Elimination System (NPDES) Lines

Determining the width of the permanent right of way is primarily a function of the typical section and drainage requirements for a section of roadway. Although these are the prevailing criterion to set right of way, additional criteria include the requirements of the NPDES. The purpose for the NPDES line is to show the contractor where it is necessary to clear and grub outside of the construction limits line, especially when the clearing and grubbing pay item is within the roadway. When the clearing and grubbing is within the right of way, then the NPDES line is only necessary when it extends outside of the right-of-way line. See the *SCDOT Requirements for Hydraulic Design Studies* and Section 12.1.5 for more detailed information on NPDES lines.

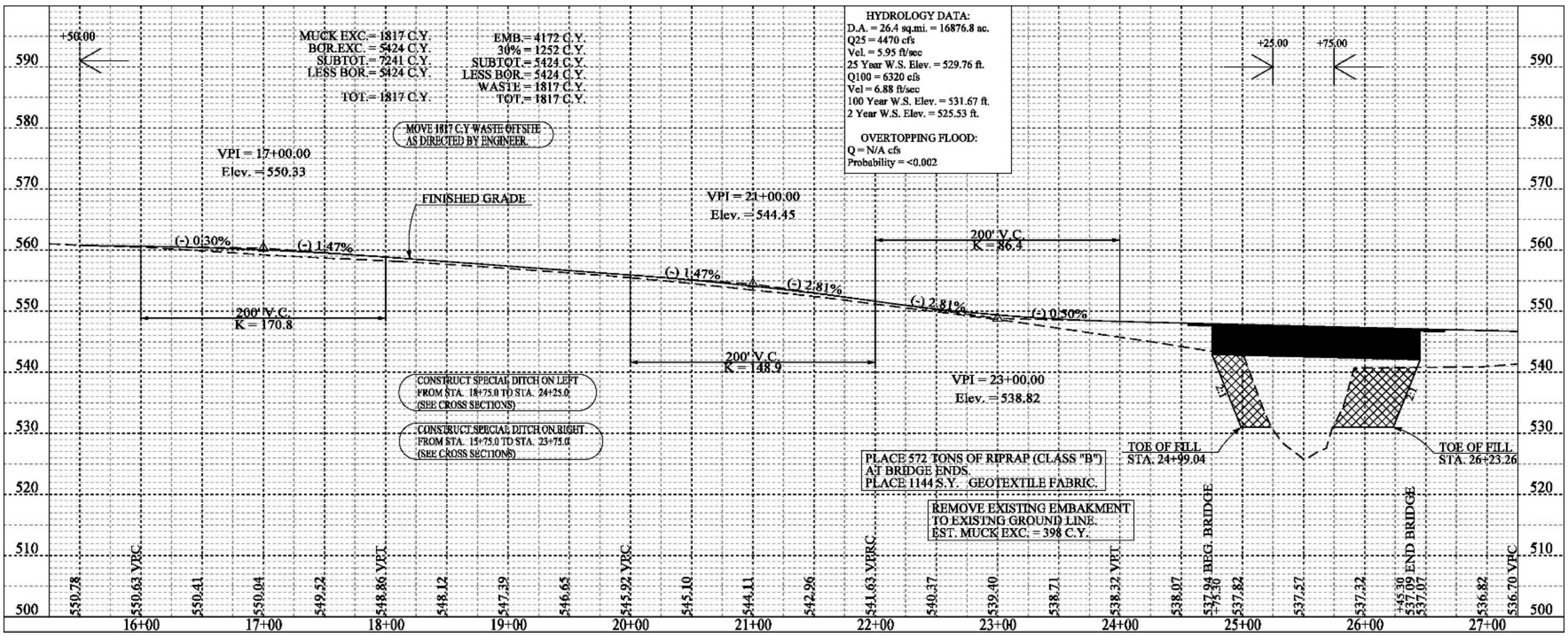
22.2.8.11 Drainage Improvements

Most highway projects require new drainage facilities and/or the improvement of existing drainage. This may be in the form of ditches, channels, streams, culverts and/or closed drainage systems.

Hydrology data is required to be placed on the profile sheet in the plans for certain drainage facilities on all projects using federal funds. This hydrology data is to be shown in detail for all box culverts, bridges and pipe culverts 48 inches or larger.

Hydraulic Design will provide the data sheets to roadway and bridge designers. The designer is responsible for adding the hydrology data in the profile area of the Plan Sheet; see Figure 22.2-F. To ensure that the correct information is placed for these drainage structures, cells have been created for the hydrology data required for bridges, pipe and box culverts and large box culverts. The cells can be found in the cell library.

It is the designer's responsibility to correctly document all planned improvements. For open drainage conditions, it is generally sufficient to show the required information next to the proposed feature. Show all storm drainage information (e.g., length, size, type (if different from typical), slope of pipes) in a table format. Indicate the location by station of drainage appurtenances together with type of structure and flow invert elevations (in/out). Culvert identifications include type, size, length and elevation inverts (in/out). Ditch and channel improvements may require side slope and bottom width dimensions and elevations.



HYDROLOGY DATA
Figure 22.2-F

22.2.8.12 Existing Ground (Pavement) Line

Once the final alignment is determined, the designer should show the vertical location of the existing ground or pavement on the profile. The existing vertical alignment is noted as existing ground. The ground line should show all elevation breaks to the degree known and, more specifically, the low points including stream, creek and river bottoms. Also, delineate the top and bottom elevations of the banks of streams, creeks and rivers.

22.2.8.13 Reclaiming Existing Roadway Notes

When using reclaiming existing pavement notes, the designer should consider the following:

1. Existing Pavements. Removal and Disposal of Existing Pavement will be measured and paid for by the square yard in accordance with the *SCDOT Standard Specifications*. Where the area to be removed goes beyond the construction limits, identify the area on the plans and include the quantities in the inclusions on the General Construction Note Sheet.

2. Existing Asphalt Pavements. Removal and Disposal of Existing Asphalt Pavement will be measured and paid for by the square yard. All existing asphalt pavement to be removed less than 2 inches will be paid for as Unclassified Excavation. On the Plan Sheet identify all asphalt pavement to be removed with an arrow and the note:

   Remove and Dispose Existing Asphalt Pavement

   Show the asphalt pavement to be removed outside of construction limits with cross hatching and include the quantity with the inclusions on the General Construction Note Sheet with the following note:

   All removal and disposal of existing asphalt pavement will be measured and paid for as described in the special provision.

3. Bituminous Surfacing. Areas where the existing bituminous surfacing (not on a stone base) is to be scarified and shaped to drain are not measured and paid for separately, but are to be included in other items of work. Where it is necessary to remove existing bituminous surfacing that is on a stone base, removal of the bituminous surfacing and base is measured and paid for as Unclassified Excavation. Include the quantities on the General Construction Note Sheet with a note referring to the special provisions for the method of measurement and payment. The plan sheet should identify the area of bituminous surfacing to be scarified and shaped, or removed with an arrow and note:

   Scarify and shape existing bituminous surfacing.
   ______________________________*No separate pay for this work

   or

   Remove and Dispose existing bituminous surfacing and stone base.
   ______________________________*Pay Item Number: 2031000

4. Existing Earth Roadway. Where it is necessary to reclaim the existing earth roadway, show the area outside the construction limits to be scarified, graded to drain and seeded with crosshatching. Identify the crosshatched area on the plan sheet with an arrow and the following note:

Reclaiming Existing Earth Roadway

22.2.8.14 Earthwork Balance

Include the balance of earthwork on the Profile Sheets for projects when Unclassified Excavation and/or Borrow Excavation pay items are used (i.e., not site excavation). Show the balance points to the nearest tenth of a foot. Typically, the limits of haul for each balance should not exceed 3,000 feet (the free haul limit). Overhaul will be paid when 3000 ft is exceeded per SCDOT *Standard Specifications*.

For detailed information on balancing earthwork, see Section 20.2 and Figure 22.2-F.

22.2.8.15 Environmental Considerations

The designer must ensure that all commitments made in an approved environmental document are included in the Construction Plans (e.g., noise walls, detention ponds, wetlands protection).

Environmentally sensitive areas may be identified on the Plan and Profile Sheet. Use the notes column on the Right of Way Data Sheet to identify properties where hazardous waste or underground storage tanks (USTs) are located. This ensures that they are properly considered during the right-of-way appraisal and negotiation process.

Underground storage tank studies are performed by the Rights of Way Office. If USTs are to be located on a tract affected by design, notify the Project Manager

22.2.8.16 Traffic Information

Future traffic data (e.g., turning volumes) for all major intersections is shown on the Intersection Detail Sheet, if available.

22.2.8.17 Railroad Information

For projects with railroad involvement, contact the SCDOT Railroad Projects Office early in the Project Development Process for guidance.

Include the following information on the plans for all projects with roadway and railroad interfaces:

- railroad right of way;
- owner of all tracks;

- when railroad tracks and roadway cross at-grade or grade separated, identify railroad milepost at crossing;
- when railroad tracks run parallel to the roadway, identify the railroad milepost at two locations within the project limits (this will identify the direction of milepost);
- USDOT crossing number;
- top of rail elevations on profile;
- for at-grade rail/highway crossings, finished roadway grade to match top of the rails (in some instances, the rails may require adjustments if requested by railroad company);
- railroad appurtenances;
- existing drainage facilities;
- new drainage facilities (coordinate with the SCDOT Railroad Projects Office/Railroad Company to determine specific design requirements);
- dimension horizontal offset to existing and/or future curb, median, or sidewalk from tracks (coordinate with the Railroad Projects Office/Railroad Company to determine specific design requirements);
- top of all rail elevations, existing and future, on cross sections; and
- railroad and roadway right of way on cross-sections when tracks parallel the roadway.

For roadway grade-separated crossings over railroad tracks, also include the following:

- profile on the survey centerline beneath the bridge;
- if identified, location of proposed future track;
- horizontal clearance to the adjacent piers measured from the centerline of existing rails and future rails, if identified; and
- vertical clearance measured from top of high rail to lowest point of superstructure (offset from centerline of track varies by railroad company; is determined through early coordination).

Reference the SCDOT *Bridge Design Manual* for additional guidance concerning highway bridges over railroads.

The roadway designer should generate the following information for Railroad Company use only. Do not include this information in the final roadway plans:

- For perpendicular railroad crossings, provide cross-sections perpendicular to the railroad at 25-foot increments for a distance of 200 feet left and right of the roadway centerline.

- When railroad tracks run parallel to the roadway, provide cross-sections perpendicular to the roadway at 25-foot increments.
- Label cross-section sheets as “FOR RAILROAD USE ONLY” by placing this note in the upper right of the cross-section sheet.
- No profile for the railroad alignment is required; however, it may be requested by the Railroad Company. When requested, provide vertical alignment on a profile sheet and label it as “FOR RAILROAD USE ONLY”.

Railroad Companies will request additional information as deemed necessary.

22.2.8.18 Bridges

Bridges should be accurately drawn on plan sheets and flagged with a note that shows the length, width and type (e.g., precast, prestressed, reinforced concrete).

New right of way should be 75 feet on each side of the centerline and extended for 75 feet beyond the beginning and ending stations of the bridge. This may be varied at the discretion of the Program Manager.

Include all applicable guardrail notes.

The profile should show the bridge thickness with an elevation shown at both ends of the bridge. In addition, show the toe of fill stations and slopes of fill under the bridge. If riprap is to be placed along the toe of fill, include the notes for the riprap. Also, include the hydrology data and high watermark. Omit the earthwork from the toe of fill to toe of fill.

In some cases, it may be necessary to remove portions of old fill. Show this by cross-hatching and itemize as Unclassified Excavation.

22.2.8.19 Profile Sheets (Sheets 6A, 7A, etc.)

Where the project is complex (e.g., urban projects, interchanges), it is often desirable to provide separate sheets for the plan details and profiles to adequately show the details. In addition, separate profile sheets may be required for side roads and other approaches if significant construction is taking place on these facilities.

22.2.8.20 Drainage Plan Sheets (Sheets D1, D2, etc.)

Where the project is complex, the designer may elect to provide separate plan sheets for the drainage/utility layouts. This will allow the contractor to adequately determine the details for these features. These are typically reproducible copies of the plan sheets or may be individually produced sheets at a scale commensurate with the extent of detail required.

22.2.8.21 Miscellaneous Items

The designer should ensure that all Plan and Profile Sheets contain the following information:

- north arrow, generally placed near the upper right corner of the sheet;
- survey and construction center lines;
- bearings;
- station equation and tie equalities;
- revision blocks;
- properly completed sheet title boxes; and
- P. E. seal or "For Information Only" cell

22.2.9 Details of Plan Sheets (Sheets 12, 13, etc.)

Support drawings in the form of plan sheets for blow-ups, top of curb elevations, centerline staking plans, etc., are to be incorporated in the plans, as necessary. Number these sheets consecutively beginning with a number one higher than the last Plan and Profile Sheet.

22.2.9.1 Top of Curb Profiles

For all projects proposing curb or curb and gutter, provide a profile along the top of the curb on separate Curb Grade Sheets. Delineate these curb and gutter profiles as curb grades and show in the same manner as for vertical alignments discussed in Section 22.2.8.5.2. The designer has the option to include the top of curb profiles as part of the Reference Data Sheets by showing the VPI data with vertical curve lengths, instead of profile sheets. Where sidewalks intersect the mainline, extend the profile plot across the intersection along the mainline edge of traveled way and show as a dashed line. In instances where side roads intersect the mainline and do not have curb and gutter, it is not necessary to plot profiles along curb radii unless there are indications of drainage problems.

22.2.9.2 Interchange Layout Sheets

For interchange improvements, it is necessary to supplement the standard plan data with additional information that clarifies the intent of the design and clearly depicts the relationship of all elements of the interchange. This is best achieved through the addition of reduced scale (no smaller than 1 inch = 200 feet) geometric layout drawings and reduced scale grading and drainage drawings adjusted to the scale necessary to depict the interchange layout. Consider using the following sheets for interchanges:

1. Interchange Geometric Layout. This drawing depicts the overall geometrics of an interchange and how each element of the improvement relates to the mainline and crossroad(s).

2. Miscellaneous Profiles. As a result of developing full-size plan drawings in interchange areas, it is sometimes necessary to develop separate profile sheets for ramps and connector roads; see Section 22.2.8.5.2. If the interchange plan shows the full extent of

work for crossroads, side streets, frontage roads, etc., their respective profiles may be shown on separate sheets.

Where structure separations are proposed, show them graphically at their proper location on the profile, including thickness of structure.

3. Grading and Drainage Plan. Conventional drawings (e.g., Plan and Profile Sheets, Cross Section) and other detail sheets cannot always include the full intent of the proposed work. For this reason, it may be beneficial to incorporate a Grading and Drainage Supplemental Plan Sheet, see Section 22.2.8.11. This drawing should clearly indicate the overall intent and extent of work and should prevent the designer from overlooking potential problem areas not readily delineated by the cross sections.

   Proposed grading work is delineated by a maximum 5-foot solid contour line for all internal areas of the interchange quadrants. External areas do not need to be contoured. Generally, these areas are sufficiently delineated by the cross sections. The existing contours are shown on the Layout Sheet only to the extent that they tie into the proposed contour system.

   If not included on the other drawings, show all proposed drainage in full detail.

   Show cross-section match lines from one roadway element to another.

22.2.9.3 Staking Plan Sheet

For complex projects, designers may develop a Staking Plan Sheet. This sheet should show the minimum data (e.g., centerline layout, major traverse control points, coordinate values of critical points and monument descriptions) and may be expanded to show all centerlines, curve data, coordinated points and survey baselines. This drawing is developed to clarify the plans and can be used by field construction personnel as an easy reference to check the horizontal geometry; see Section 22.2.8.5.1.

22.2.10 Sequence of Construction and Traffic Control Plan (Sheets TC1, TC2, etc.)

On non-complex highway projects, the construction sequence and control of traffic is easily discernable and any needed clarification is shown by use of plan notes. As projects become more complex, the need arises for additional clarification through the development and inclusion of both the construction sequence and control of traffic sketches and notes.

22.2.10.1 Work Zone Traffic Control Sheets

The Traffic Engineering Division has developed a set of typical sheets and construction notes that may be used on roadway projects that are specific to the control of traffic during construction. The designer is cautioned to carefully review these sheets for applicability, completeness and clarity as they pertain to each specific project.

22.2.10.2 Work Zone Traffic Control

Traffic control drawings should be incorporated into a schematic layout of the improvement and include intersecting roads and streets for the purpose of depicting the location and type of permanent construction signs to use during construction. Permanent construction signs must be quantified and included in the Summary of Estimated Quantities Sheet.

The construction sequencing and temporary traffic control should not be left until the later stages of plan completion, but should be considered in preliminary planning and continually refined through the entire plan development process.

On complex projects (e.g., urban settings that involve existing roads and streets), the construction work accomplished for each phase of project development must be clearly identified on the plans. The designer may develop multiple reproductions of the project Plan and Profile Sheets to achieve this goal. Using uncluttered drawings that depict only the existing conditions and delineating the new work thereon for each phase until all work is completed is another method of achieving this goal. The designer should include major work items (e.g., the removal and demolition of pavements, box culverts, bridges) as required in each phase of the work.

Most traffic control items (e.g., signs, cones, barrels) required by the contractor during construction are addressed in the *SCDOT Standard Specifications* and are accounted for in the Summary of Estimated Quantities Sheet under the pay item “Traffic Control.” Temporary concrete barriers, used for the control of traffic during various construction phases and stages, must be quantified separately on the Summary of Estimated Quantities Sheet as a pay item. Consult with the traffic designer to determine its current practice of payment for work zone traffic control items. Major projects (e.g., the improvement of freeways and arterial highways) may require more elaborate Traffic Control Plans. This needs to be coordinated with the traffic designer.

22.2.10.3 Detour Plans

Traffic detours may be implemented through the use of existing roads, construction of additional facilities, or a combination of both. If the detour is entirely over existing routes, this route and accompanying detour signs are shown on the Work Zone Traffic Control Sheets.

If existing routes used for detours require pavement overlays and/or widening, show the required work on the mainline Plan and Profile Sheet, a reproduction of the mainline Plan and Profile Sheet, or as a separate drawing.

Detours requiring new construction necessitate that the alignment geometry be shown and developed on a separate Plan and Profile Sheet. This sheet can be created from a reproduction of the mainline Plan and Profile Sheet, for all or any portion of the detour that can be shown, or by producing a new sheet and developing the detour geometric design details.

It will be necessary to quantify all pay items required to construct the detour as well as those pay items necessary for its removal and restoration.

22.2.10.4 Work Zone Control Plan and Detours

For complex projects, Design Field Review prints should include a phased plan for maintaining traffic during construction. The sequence identifies where traffic is relocated during various phases of construction that encroach on current roadway locations. This plan is a combination of small-scale drawings identifying areas of construction and showing the location of traffic during the respective construction encroachments. The drawing should be accompanied by a brief narrative description of construction activity and concurrent traffic operations. A complete narrative will be developed later as a special provision and incorporated into the Construction Package. The Work Zone Traffic Control Plans should describe the types and locations of traffic control devices and signs used at each phase of traffic relocation if different from the normal practices contained in the *Manual of Uniform Traffic Control Devices (*MUTCD*)* or *SCDOT Standard Drawings*. The plan will identify lane widths, transition taper lengths and any geometry necessary to define temporary roadway alignments. The plan should also address the type of surface used for all temporary roadways.

22.2.11 Electrical and Lighting Plans (Sheets E1, E2, etc.)

Although lighting design may be incorporated into final Construction Plans, a section addressing lighting specifications is not presently included in the Department's *SCDOT Standard Specifications*. However, a sample special provision for lighting may be obtained from the Traffic Engineering Division. When packaged as part of the Construction Plans, all design drawings may be prepared in the standard roadway sheet format. A complete lighting design package includes plans, quantities and specifications necessary for bidding purposes. If development of these plans and specifications are required, the traffic engineer is responsible for preparing these plans. However, the roadway designer is responsible for incorporating the quantities onto the Summary of Estimated Quantities Sheet.

22.2.12 Landscaping Plans (Sheets L1, L2, etc.)

When landscaping items are incorporated into the plans, they may be delineated on the standard Plan and Profile Sheets, reproducible copies of the Plan and Profile Sheets, or individually produced sheets at a scale commensurate with the extent of detail required.

The location and limit of all single species of trees, plants, shrubs, etc., and plant groups and beds are clearly indicated on the plans. The plans include all special details necessary for placing, supporting, mulching, fertilizing, trimming, etc., of the plants.

Provide and list numbered landscaping items and quantities on the landscaping plan sheets. Do not show landscaping items on the Summary of Estimated Quantity Sheet. Enter the items and quantities into WebTransport using the pay item for each item listed on the landscaping plans. Each pay item requires a supplemental description and matches the numbering system shown on the Landscaping Summary contained within the plans.

- Include a summary of all landscaping items on the last sheet of the landscaping plans. List and number all items on the plans and enter into WebTransport.

- If multiple projects containing landscaping are let together, then the list of landscaping items on the summary sheets are numbered consecutively to ensure that no two landscaping items have the same item number within the supplemental description in WebTransport.

- Include a breakdown of the quantities that are needed for the stations or areas of the project on the additional landscaping plans. Do not include the landscaping item number on the additional sheets.

22.2.13 Pavement Marking Plans (Sheets PM1, PM2, etc.)

Pavement Marking Plans are developed in accordance with the *Manual on Uniform Traffic Control Devices* and the *SCDOT Standard Drawings* for pavement markings and raised markers. The Pavement Marking Plans are developed by the traffic designer. The layout sheets should show all lane lines, edge lines, centerlines, markings for passing and no passing situations (the limits of which will be determined in the field by the Resident Construction Engineer), stop lines, channelizing lines, pavement arrows and pavement markers. Notes that can help clarify the plans are encouraged. The traffic designer is responsible for the development of the pavement marking design and associated quantities. The roadway designer is responsible for incorporating the item quantities for both permanent and temporary pavement markings onto the Summary of Estimated Quantities Sheet.

22.2.14 Signing Plans (Sheets SN1, SN2, etc.)

When packaged as part of the Construction Plans, the designer should ensure that all designs and drawings are prepared on the standard roadway sheet format. A completed signing design includes plans, specifications and quantities for bidding. If development of these plans and specifications are required, the traffic designer is responsible for preparing these plans. However, quantities from these sheets are included on the Summary of Estimated Quantities Sheet by the roadway designer.

22.2.15 Traffic Signal Plans (Sheets TS1, TS2, etc.)

When incorporated in the Plans, traffic signals are developed in accordance with the *Manual on Uniform Traffic Control Devices* and all details of the design are presented on the standard roadway plan sheet. The traffic designer is responsible for the development of the Traffic Signal Plans and all associated quantities. The roadway designer is responsible for incorporating traffic signal quantities in the Summary of Estimated Quantities Sheet. Verify that adequate right of way is obtained for signal equipment including guy wires.

22.2.16 Roadway Structure Plans (Sheets S1, S2, etc.)

Roadway structures consist of non-bridge type structures (e.g., retaining walls, footings, foundations, box culverts, pedestrian overpasses, buildings for weigh stations, rest areas). These items may appear in the plans with or without bridge improvements and are typically included with the Construction Plans.

Normally, the structural designer will develop these structure plans. However, if these plans are being packaged in the Construction Plans, design drawings are prepared on standard roadway sheets with title boxes and all work items are quantified on the Structure Summary Sheet and included on the Summary of Estimated Quantities Sheet.

22.2.16.1 Drainage Structures

The Right of Way Plans reflect the intent to which drainage impacts the proposed right of way and parcels of land to be acquired. To this extent, the major drainage items are established and documented on the plans. It is mandatory that the structure type be determined and depicted on the plans in cases where the roadway overpasses a creek or stream (e.g., bridge vs. box culvert). The same applies for parallel roadway drainage, ditches and channels, conveyance of storm drainage at side roads and driveways and clean out requirements of transverse ditches and streams.

Show the locations of all drainage system structures (e.g., inlets, headwalls, wingwalls, manholes, junction boxes) graphically with the location identified to the station offset and invert elevation. The pipe size and material type are also shown in the plans by developing a drainage table.

Where drainage structures require inlet, outlet or special treatment are not covered by the *SCDOT Standard Drawings*, prepare a special detail drawing. Show all pertinent dimensions including length, width, thickness, size and location of all concrete and reinforcing steel. Special details should reference the *SCDOT Standard Specifications* pertaining to the use of materials and construction procedures.

Drainage areas are to be shown for pipes 48 inches or greater. Include the hydrology data in the details for all culverts 48 inches or greater on Federal-aid projects.

The sectional view is a tool for setting the limits of drainage structures. Inlet and outlet structure locations and invert elevations are established and shown with respect to the roadway shoulder slopes and the existing stream profile. Pipes with a diameter greater than 36 inches will not fit in the standard 4x4 box. Show any approach and discharge stream grading or special treatments in the sectional view along with a transverse section of the channel improvement. For the transverse approach and discharge channels, show the bottom width, side slope, design stormwater elevation and type and thickness of lining.

Show the hydrology data on the drawings in all cases incorporating bridges/culverts conveying water through a roadway section. Data should be incorporated as recommended in the *SCDOT Requirements for Hydraulic Design Studies*.

Sideline pipes are identified as longitudinal pipe culverts in roadway ditches at driveway and other locations. Section 20.5.2 provides the guidelines for establishing pipe lengths for drives.

22.2.16.2 Other Structures

Right of Way Plans should show other structures (e.g., retaining walls) in the plan and cross section views. Include all information that relays the intent of improvements.

Additional right of way should be shown for overhead sign structure footings, if required. Conventional signing should also be checked for additional right of way requirements.

22.2.17 Plan Sheets for Ground Improvement Methods (G1, G2, etc.)

As applicable, plan sheets for ground improvement methods will be included in the plans. The geotechnical designer will coordinate with the roadway designer to develop plan sheets for ground improvement methods as required by the *SCDOT Geotechnical Design Manual*. Insert these plan sheets immediately after Roadway Structure Plans and number them as G1, G2, etc.

22.2.18 Erosion Control (EC1, EC2, etc.)

The application of temporary erosion control items is generally left to the discretion of the Resident Construction Engineer. However, the hydraulic designer should carefully review the project and determine if specific applications are required and, if so, they should be shown on the standard Plan and Profile Sheets or reproductions. If necessary, the plans should show the proposed method of controlling drainage runoff during the various stages of construction so that neighboring streams, bodies of water and properties are not filled, contaminated or covered by siltation originating from the project construction area. Erosion Control Plans should identify the location and type of measure used to control erosion and sedimentation. The methods may require silt fences, detention ponds, temporary seeding, ditch checks, filtration dams, etc. Ensure quantities are included to clean and maintain erosion control measures as the project progresses. Also address removal of temporary erosion control devices following construction.

During the Design Field Review, the hydraulic designer and Resident Construction Engineer will determine the need for a specific erosion control plan.

If additional or specific erosion control items are required, clearly indicate and properly quantify these items in the inclusions for erosion control. If reproductions of the Plan and Profile Sheets are used, number all Erosion Control Plans using the EC series.

Pay items to construct these sediment and erosion control items are described on each drawing. A unique pay item number is available for each.

For additional information on erosion control, see Section 20.5.3 and the SCDOT Stormwater Management internet site.

22.2.19 Utility/Utility Relocation Sheets (Sheets U1, U2, etc.)

Not all plans will have plan sheets provided by utility companies showing their final locations. The Utility Relocation Sheets show the future locations of all utilities, including those that will remain in-place. Include the following note on these sheets, if the work is not included with the contract:

– For Information Only – It is the responsibility of the Contractor to coordinate with the Utility companies for final utility locations.

Include utility owner information, SUE details and other existing utility information as needed for the project. If plan sheets are used, the information unrelated to utilities may be dithered to improve the readability of the utility line styles and other pertinent information.

22.2.20 Utility Construction Sheets (Sheets UC1, UC2, etc.)

When Utility Construction Sheets are provided, number them using the U series (U1, U2, etc.) and bind separately. These sheets are used when a specific utility company provides drawings for the construction of utilities within the roadway construction limits and this work becomes part of the highway contract. These plan sheets will be numbered by the Utility Office as discussed above, then forwarded to the roadway designer to be processed with the roadway plans for a highway letting. The Utility Construction Sheets are not to be included in the roadway plans.

22.2.21 Cross Sections (Sheets X1, X2, etc.)

22.2.21.1 General

The required frequency of cross sections depends, in part, on the type of highway project and the terrain. The purpose of a cross section is to:

- establish reasonable estimated earthwork quantities,
- show foreslopes and back slopes,
- show non-standard ditches,
- show cross slopes, and
- show guardrail.

Plot the sections progressively in the direction of the increasing stationing beginning at the bottom of each individual Cross Section Sheet.

Final cross sections should reflect existing and proposed roadway templates, slopes as appropriate, superelevation information (except where curb and gutter prevails), earthwork areas and volumes, major drainage items, cross line pipes, station equalities, bridge station limits, major drives, intersecting roads, guardrail, mucking limits and replacement materials and volumes and other items that clarify the intent of the drawings. See the sample sheets on the SCDOT Roadway Design internet site for example cross sections.

22.2.21.2 Existing Ground Lines

Ground lines may be obtained from mapping, field surveys, or a combination of both. They should be representative of existing conditions at the exact location of the proposed roadway template. The ground lines should reflect features such as existing road crossings, retaining walls, streams, swales, ridges and buildings.

22.2.21.3 Existing Cross Sections

Cross sections are usually taken at increments of every 50 feet along horizontal curves, 100 feet along tangent sections and other points deemed necessary. Where it is anticipated that there will be deep cuts or fills, it may be necessary to space cross sections further apart. These cross sections should be plotted on a scale of 1 inch = 5 feet both vertical and horizontal. On major or arterial projects, usually, cross sections are taken 75 feet to 100 feet left and right of the centerline. This will make it necessary to place cross sections up the center of the page. In addition, if cross sections go out more than 75 feet on each side it will be necessary to change the scale to 1 inch = 5 feet vertical and 1 inch = 10 feet horizontal. Show the scale in the lower right corner within a 1-inch square block. Then show the proper scale along the vertical and horizontal lines.

Include the designated station number horizontally approximately 1 inch below the cross section on the centerline. Show the existing ground line elevation vertically on the centerline approximately ½ inch to 1 inch above the existing ground line.

All dimensioning for station, existing elevation and proposed elevation are controlled within the GEOPAK criteria file used to layout the cross sections on sheets. Show existing ground lines on cross sections and profiles with a dashed line. When existing base and surfacing is in place, diagonal lines will be shown under existing base and surfacing.

22.2.21.4 Proposed Cross Section

When plotting the new construction on the cross sections, consider the following:

1. Beginning and End Notes. Include a beginning note under the first cross section and the ending note after the last cross section. For example:

   Survey Sta. 0+10.00 Begin
   Project ID. P012345 – Road S-1028

2. Berms and Swale Ditches. Include a note at the beginning of each ditch.

   Construct Swale Ditch Lt. Sta. 1+25.00 to Sta. 7+50.00

   Include the grade and elevation of the ditch and show these dimensions above each section on the front edge line of the ditch. Special ditches 300 feet or longer require a note, along with grade and elevations, to be shown on plan and profile sheets as described for swale and berm ditches. Front ditch slopes may be steepened or extended further out to obtain positive drainage without grades and elevations when less than 300 feet in length.

3. Bridges. Provide a cross section at the beginning and ending bridge stations.

4. Superelevation. Provide a note for the beginning and ending of the superelevation transition and where the maximum superelevation begins and ends.

   Begin / End SE

   Begin / End Maximum SE

5. Proposed Template. The proposed roadway template should reflect the desired roadway cross section at the desired locations. To achieve this, the designer must show pavement breaks, shoulder breaks, foreslopes, backslopes, ditches and identify all slopes beyond

the shoulders that are non-typical (that which varies from the typical section) or control slopes. Each cross section is identified by centerline stationing and grade point elevation. It is also necessary to show proposed curbs, sidewalks, guardrail and retaining walls as these elements help to properly identify the proposed improvements.

6. Drainage. Thoroughly review each project to ensure positive drainage, templates and existing ground lines are plotted at the correct elevation and that templates are plotted accurately according to the Typical Sections. Include sideline and crossline pipes on cross sections as specified in the *SCDOT Specifications and Support Manual for Geopak Drainage*.

7. End Areas and Volumes. End areas are computed mathematically for cuts and fills at each section. Show cut areas volumes on the sheet by listing them horizontally to the left of the centerline and below each cross section. Fill areas are registered on the sheet by listing them horizontally to the right of the centerline and below each cross section. Corresponding volumes are listed vertically left and right respectively, above each cross section. Earthwork computations are performed and included as the output data in project file for future reference. Provide the end area horizontal on the 10-foot line (1 inch = 5 feet scale) or 20-foot line (1 inch = 10 feet scale) with the cut shown on the left and the fill on the right. Show this at the same level as the station number. Show the earthwork volume vertically on the 15-foot line (1 inch = 5 feet scale) or 30-foot line (1 inch = 10 feet scale) and place the quantity between the stations. Show the volume for the first cross section on the first page below that cross section and include the volumes for the first cross section on the following pages at the top of the preceding page.

   For more detailed information on end areas and volumes for cross sections, see Section 20.2.

8. Clarification Data. Additional data in the form of notes is extremely beneficial during the construction phase of a project. The designer should incorporate the following information:

   - bridge limits via begin and end stationing;
   - limits of dumped stone, hand placed riprap and mucking;
   - location of existing right of way at and beyond the beginning and ending points of a project;
   - pavement removal notes and stationing for superelevation; and
   - outlet ditch, channel and other drainage information.

9. Other Items. Show guardrail and closed drainage, special boxes, etc., on the cross section.

10. New and Existing Right of Way. Show new and existing right of way.

11. Railroad. See Section 22.2.8.17 for guidance.

22.2.22 Bridge Plans (Sheets BR1, BR2, etc.)

These sheets only apply to Bridge Plans that are included as part of the Roadway Plans, which is not typical. Bridge Plans are typically bound separately.

Bridges should be accurately drawn on plan sheets and flagged with a note that shows the length, type (e.g., precast, prestressed, reinforced concrete) and location.

Ensure all applicable guardrail notes are included.

The profile should show the bridge thickness with an elevation shown at both ends of the bridge. Also, show toe of fill stations and slopes of fill under the bridge. If riprap is to be placed along the toe of fill, include a note identifying the riprap. Provide the hydrology data and high water mark. In most instances, earthwork should be omitted from toe of fill to toe of fill; however, in certain cases, the designer may need to consider it beyond the toe of fill.

In some cases, it also may be necessary to remove portions of old fill. Show this with cross-hatching and estimate as Unclassified Excavation.

When Bridge Plans are to be incorporated into the Construction Plans, the designer should ensure that all drawings are numbered in accordance with the bridge series. The roadway designer is also responsible for ensuring the quantities from the Bridge Summary of Estimated Quantities Sheet are included in the Road Design Summary of Estimated Quantities Sheet.